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Sir Hubert Walker, C.B.E., M.I.C.E., Chairman of the Division, in the Chair

The following Paper was submitted for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Airport Paper No. 18

"Gravel Compaction and Testing, and Concrete Mix Design at London Airport"

by

Harold Smith, A.M.I.C.E.

SYNOPSIS

The Paper is intended to show the practical interpretation, in the field, of the results of laboratory experiments. The experiments at London Airport were carried out either in the field laboratory or on a full-scale test on the site. The laboratory experiments were of a type to suggest a particular trend in the behaviour of the variables in soil compaction or concrete mixes, but the final results were obtained in the field.

The Paper opens with a brief description of the geological structure of the drifts in and around London Airport and gives details of the methods adopted in the testing of the gravel subgrade and experience gained in the study of the compaction of gravel. Rapid methods of testing are described, together with sufficient data to enable a simple and economic control to be maintained.

Details of concrete mixes used on the Airport are given and it is shown that a lean mix can be stronger than a rich mix, having the same water/cement ratio. It is suggested that there is a need for further research into the problem of optimum water for mixing. It is shown that there is only one water/cement ratio for a concrete mix and that the generally accepted rule that the water/cement ratio governs the strength appears to follow a definite law only when using "optimum water."

The Paper deals with a simple and practical way of designing and controlling concrete mixes based on analytical methods which can be carried out in a field laboratory.

The method of placing concrete is shown to be the main control in the design of a mix and that control by direct supervision on site is not necessarily the most economical way to produce good-quality concrete.

The method used in determining the three main controlling limits of a concrete mix is fully described and shows that the use of a mix having a slump is fundamentally wrong, and that workability is controlled by the grading and shape of the aggregate and not by the addition of water.

The relative merits of volumetric and weigh batching are discussed and an attempt is made to bring them into a proper perspective. In certain circumstances weigh batching can be more variable than volumetric batching and the limits of both methods are briefly discussed.

INTRODUCTION

SOIL compaction and concrete-mix design are subjects amply covered by books and Papers but the application, in practice, is left to the engineer, who, more often than not, follows literally the written word and obtains unexpected results.

The Author has frequently discussed with other engineers their problems in connexion with soil compaction and their difficulties have always been caused by too rigid adherence to laboratory and text-book methods applied in the field.

It is frequently stated in text books that the flat smooth steel roller gives the best results for compacting gravel. Optimum moisture for gravels is usually given as about 7 per cent with dry densities of about 138 lb. per cubic foot. Other types of rollers are stated to be inefficient on gravels and always give lower results. Whilst that might be true on relatively small sites it does not follow that it is the most economical and speedy way of compacting gravel on a very large contract. It would not be economically possible, in the English climate, and on a large contract the size of London Airport, to compact gravel with a flat roller.

For example, in a project of this size it would be far too costly to attempt to roll gravel at 7 per cent optimum moisture with a flat roller, or, for that matter, at 7 per cent optimum with any roller. From the sieve analysis of the gravel used at the Airport, the natural moisture content was about 4 per cent, and rolled quite easily to 138 lb. per cubic foot with pneumatic-tired rollers at that moisture content.

To attempt to increase the moisture from 4 to 7 per cent would have been impracticable and costly, because the daily quantities of gravel laid were of the order of 4 to 5 acres.

Experience at the Airport showed that flat rollers were only suitable for thin layers of gravel not exceeding 4 inches, and if the moisture content was over 5 per cent the surface became sealed and a spongy condition was set up which was difficult to eliminate.

The large areas of gravel prepared by the contractor, each day, required immediate testing and the whole of the contractor's operations depended upon rapid decisions to enable him to organize his plant, and to attempt to test such large quantities by laboratory methods would have been slow and cumbersome.

It is generally suggested that ten density holes per test should be the minimum and that at least one complete test should be taken every 1,000 square yards. At London Airport with over 20,000 square yards per day to be tested it was impossible to carry out such a large number of tests. It was necessary to reduce the number to reasonable limits to enable the contractor to work economically and also to organize a satisfactory control of the compaction of the gravel.

The mixing and placing of concrete suffers from handicaps similar to that of the compaction of gravel in so far as that so much has been written and said about it that engineers appear to have difficulty in arriving at a satisfactory method of control.

The control of concrete should be started in the design stage but generally too much is left to the contractor, or the engineer, to decide finally the most suitable mix. A fairly common practice in specifications is to give the complete mix either by volume or weight, including the water/cement ratio and strength. This usually leads to difficulties in the field as, frequently, the mix is unworkable because the contractor has chosen a method of placing the concrete which does not fit in with its workability.

Various suggestions are then made as to the cause of these difficulties: either the sand is too coarse or too fine, too much or too little, the water cement ratio is incorrect, the stones are of unsuitable shape, grading is wrong, workability is too high or too low, and the wrong compaction factor is being used. The tendency is always to place primary importance to one or two of these variables but, put into their proper perspective, all of the above factors have their place in the control of concrete.

If it is necessary for the engineer to adjust the mix, it is far better to use the local or chosen aggregates and re-design the mix to suit their general characteristics and not to attempt to adhere to any fixed grading curve or shape. A study of grading and shape shows how unnecessary it is to adhere strictly to any particular grading curve or shape as workability can be simply controlled once the natural character of the local supplies is known.

A too rigid adherence to theoretical curves and tables of mixes requires very strict direct supervision—a hidden cost which can be reduced or eliminated altogether if a mix is properly designed to suit the aggregates and the method of placing the concrete.

If, in placing concrete runway slabs by means of vibrating machinery, the mix is correctly designed, the control is largely automatic. The inherent difficulties of trying to measure the water contained in the aggregates leads to final control of the mix being made by eye.

Any variation in workability is immediately shown up by the vibrator and the gangs working behind the machine are either left behind or halted, depending upon whether the mix is workable or not.

Whatever the conditions, word is sent back to the mixer and a slight adjustment is made in the water to give the required workability. If the adjustment of the water is more than 2 per cent, the operator will also adjust the weight of the aggregates.

The variations in workability are usually caused by difficulty in assessing the moisture in the aggregates and changes in the grading of the coarse aggregate and the sand. The slight addition or subtraction of water has a very small effect upon the ultimate strength of the concrete.

The main cause of variation in workability is in the grading of the sand, but the moisture can be controlled to very fine limits by stock piling in large heaps and allowing to drain for at least 24 hours.

The change in grading of the coarse aggregate must be considerable to have any noticeable effect upon workability and, in any case, if there is any great change the addition of water will not help and the error mxst be rectified.

These changes in workability are more frequent in weigh batching than in volumetric batching since, in the former, any change in added water should be accompanied by a change in weight of the aggregates, which is not always carried out, but in volumetric batching it is only necessary to change the water.

The bulking of sand in volumetric batching is not the handicap that it is made out to be as the bulking curve is very flat at the moisture contents usually found in practice. The variation in sand moisture content, when large stockpiles are used, of say 500 cubic yards is about 1 per cent, ranging from 5 per cent in the summer to 6 per cent in the winter.

A more consistent mix, as regards workability, is obtained with volumetric batching, the yield of the concrete being more accurate and tending to give a cheaper product.

As regards strength, it is possible to get just as good a coefficient of variation in volumetric as in weigh batching, taking into consideration the more accurate yield obtained with the former.

At London Airport weigh batching was used successfully, since very large stock piles were used, which enabled all aggregates to drain down to a constant figure after 24 hours' storage.

In general, the only times in which weight changes had to be made were early morning and late afternoon.

The morning change was necessary because of the accumulation of water in the overhead sand hopper which had been filled over night, whilst the afternoon change was necessary because, by then, the bottom 2 feet of sand was being used when the moisture content was very high.

COMPACTION OF GRAVEL

Site Conditions

The principal gravel deposits in the Thames valley are the Flood Plain, which lies close to the river, the Taplow Terrace, which is the next outer terrace or deposit, and further out, on the rising ground, is the Boyne Hill gravel. A more recent deposit, geologically speaking, is the alluvial gravels which follow the beds of the tributaries of the River Thames.

The site of London Airport lies on the Taplow Terrace gravel deposit, as shown on the drift map in *Fig. 1.*

The general details and characteristics of the gravels are as follows.

Flood Plain.—This usually occurs on drifts of up to 30 feet depth overlying London blue clay. It is clean, well graded, and with a sand content of about 35 per cent, the sand being of medium fineness. Being near the river the water table is high, generally about 3–4 feet from ground level, and the gravel is eminently suitable for processing for concrete aggregates.

The general method of winning this material is by suction pumps on pontoons which discharge into barges, and it is taken to a shore pumping station which transfers the gravel to the screens. This gravel is very clean, with seldom more than 5 per cent of silt, and is not so suitable for compaction as Taplow or Boyne Hill.

Taplow gravel.—This gravel is found in depths of up to 20 feet overlying the London blue clay; it is not so clean as the Flood Plain and contains at least 12 per cent of silt, and sometimes more.

The grading of the stone is regular, with no more than 5 per cent retained on the 1½-inch sieve. The sand content is about 35 per cent, of medium fineness, but slightly finer than the Flood Plain.

The water table is about 6 feet from the surface and this gravel also lends itself to processing for concrete aggregates. It does, however, require more vigorous washing than the Flood Plain.

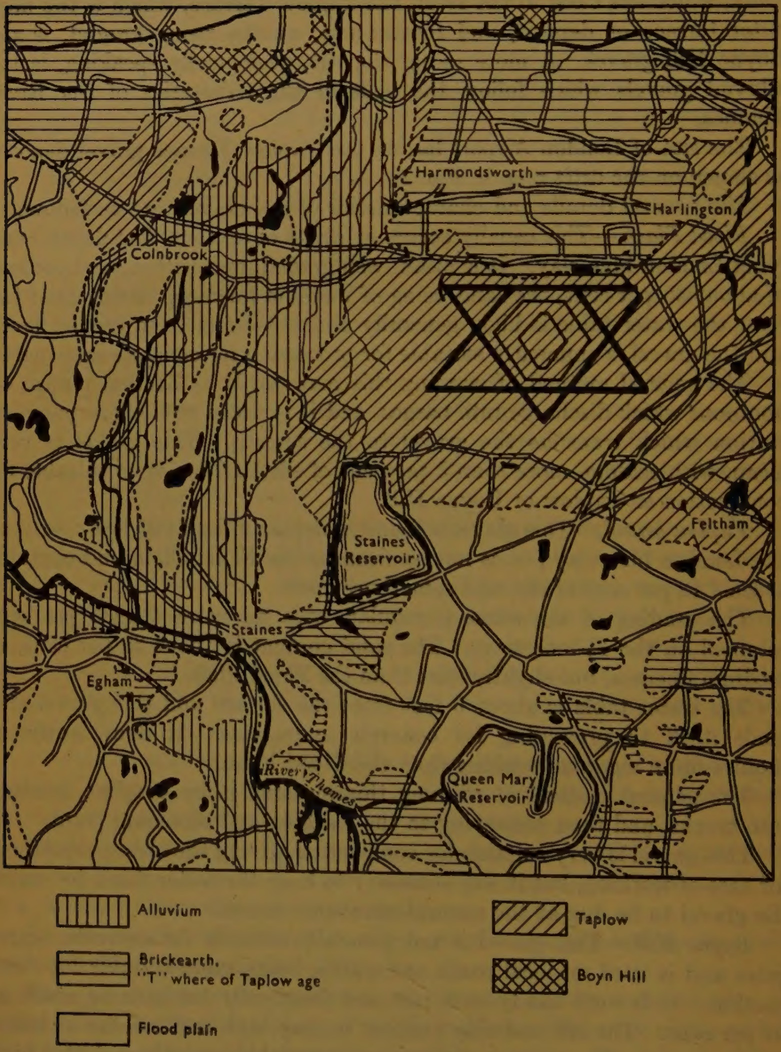
The general method of winning this material is by dragline, loading into trucks, and then conveying to the washing and screening plant.

This gravel is very suitable for compaction, giving the ideal proportions for ease of working, but it was necessary to keep the water down to enable the gravel to be dug at the natural moisture content.

Boyne Hill.—This gravel is not generally suitable for concrete aggregates and is used on local roads and paths, being more suitable for compaction. It is very sandy as a rule and frequently contains as much as 60 per cent. The silt and clay content is very high and reaches as much as 25 per cent. The grading of the stone is variable, sometimes with a high percentage over 2 inches and often quite the reverse with the maximum size no greater than ¾ inch.

The Boyne Hill terrace, being higher, seldom contains any ground water and can be easily worked in the dry.

Fig. 1



Drift Map of Gravel Deposits

Alluvial deposits.—These deposits are very shallow, seldom more than 10 feet deep, and average about 5 feet.

It is clean and the sand sharp and coarse, whilst the stone is well graded. The percentage of sand is very low, usually in the region of 25–27 per cent, and although this gravel is worked for concrete aggregates it generally produces a harsh concrete and is not suitable for compaction.

It can be seen from this description of the various gravel deposits that the site of the Airport is on an eminently suitable subgrade. All gravel used for compaction was obtained from borrow pits inside the Airport boundary.

When the site was taken over in 1944 there was approximately 80 acres of working and disused gravel pits. Those pits were up to 20 feet deep, with at least 14 feet of water in them and large quantities of silt which had been deposited back from the washing of the processed gravel for concrete.

The formation for the runways, tracks, and standings was on either natural or compacted gravel, and as some of the runways crossed the disused pits it was necessary to pump them dry, clean out the silt, and refill with compacted gravel.

The specification required that the gravel was to be laid in layers not exceeding 9 inches in thickness when compacted, and to fill these pits over 2,000,000 cubic yards of gravel and sand was used.

For the remainder of the site the top vegetable soil was stripped off and spoil heaped, then the overburden of brick earth, approximately 3 feet thick, was removed and, if necessary, the formation was brought up to level in compacted gravel.

The output of compacted gravel was of the order of 40 acres per week, with about 8 to 10 acres of finished and trimmed formation between concreting forms.

All this gravel had to be inspected and tested for density and no further layers were permitted until the contractor had a clearance certificate.

With this quantity of gravel being handled each week rapid and effective methods of testing had to be devised.

In the first instance the source of borrow material was tested and the Field Engineers and Inspectors informed of the probable density to be achieved and the optimum moisture to aim at.

Areas of gravel laid were of the order of 600 feet by 150 feet at a time, on which the contractor would take about twenty density tests. If he thought his tests were satisfactory he would notify the Air Ministry Testing Engineer of the results and a check test would be made.

At the commencement of the contract it was thought that the best method of compaction would be to use sheepsfoot rollers until they " walked out " about 4 inches and then compact the remaining loose gravel with rubber-tired rollers.

In a later contract only 8-inch layers were specified and in view of the high degree of compaction obtained with the rubber-tired rollers it was decided to try them without using the sheepsfoot roller. The result was unsatisfactory since a wavy formation was produced, owing to difficulty in eliminating the effects of dumping the gravel from tip lorries.

The final method was to give two or three passes with the sheepsfoot roller to even out the effects of dumping and then finish with the rubber-tired roller, but eventually the sheepsfoot roller was dispensed with altogether.

The last layer was arranged to come within about 2 inches of formation level, after which the concreting forms were fixed and the final 2 inches of gravel combed and power-rolled to a smooth finish.

The inspection and testing between the forms was more rigorous than for the general layers, and after inspection all apparently weak places were tested and removed or re-rolled if they failed to pass the minimum density.

The final placing of this last 2 inches was possible only between April and September and during the winter months dry lean concrete, 14 : 1 by weight, was used instead of gravel, except in the dry winter of 1949–50.

Methods of Testing and Control

In view of the large areas of gravel which had to be tested, simple methods were devised for testing and the determination of optimum moisture to enable the contractor to carry out his operations as economically as possible.

The variations in grading of the gravel were known from samples taken from the pit before operations commenced and for field work the optimum moisture can be calculated by allowing 1 per cent for the stone, 9 per cent for the sand, and 15 per cent for the silt. These figures are what might be termed the natural moisture content and the method of finding them is given later in the Paper. It will be found that they are sufficiently accurate in practice, as, in any case, it is unnecessary to try to raise the moisture content of, say, 7 acres of gravel, daily, to within 1 per cent of the optimum. It is not a practical or economical proposition to try to accomplish laboratory results in the field.

An average sieve analysis is of the order of 60 per cent stone, 35 per cent sand, and 5 per cent silt; with these percentages the optimum is 4·5 per cent, and in general the gravel was taken from the pit at about this moisture content, although the water level had to be kept down by pumping to enable the gravel to be won.

It was, therefore, unnecessary, as a rule, to add any water, provided that the gravel was rolled immediately it was deposited and spread. It was, of course, necessary to roll as soon as possible in order to seal the surface in case of rain between operations. If loosely deposited gravel became wet and above optimum, it was removed.

The usual method of finding the optimum moisture in a gravel is to sieve out the fine material and apply the Proctor density test. Having determined the optimum for the fines, an assessment is made for the stone and the two results added together. This is, of course, a laboratory operation and takes up considerable time and would seriously hold up operations in the field.

The probable density obtainable in the field can be calculated once the sieve analysis is known. It is only necessary to find the proportions of stone, sand, and silt and calculate the maximum hypothetical density by the absolute volume theory. First, the maximum bulk densities of the three component parts are obtained; the voids in the stone are then calculated, adding in the sand to determine the remaining voids, into which is added the silt; and the resulting bulk density is taken as the hypothetical maximum.

In general, the figure to allow for a practical density in the field is about 90 per cent of this maximum, and it will be found to be approximately 95 per cent of the Proctor density.

A simple way of showing the probable density obtainable is by drawing up a chart, as shown in *Fig. 2*, giving the calculated theoretical dry density based on the absolute volume theory.

Gravel can be divided, for convenience, into three main component parts—stone, sand, and silt—from which, knowing their characteristics, it is possible to calculate the theoretical maximum dry densities of any combination.

From the Taplow gravel found on site, the following figures were obtained :—

	Maximum bulk dry density : lb./cu. ft.	Specific gravity
Stone	103	2·52
Sand	107	2·60
Silt	105	2·70

The maximum bulk densities of the sand and silt were found by thoroughly ramming in a dry condition and then gradually adding water to a point past the optimum, plotting the dry densities at various intervals. It was found under those conditions that the same density was obtained for the dry condition as for the optimum. In the case of sand, the same figure of 107 lb. per cubic foot was found for the dry condition as well as the optimum and also the flood point. A similar condition was found with the silt.

In the case of the stone, if a field moisture test is taken in a very large heap, it will be found that after a short time from flooding a constant moisture content is obtained. This can be termed the natural moisture content.

The figures given previously for the three component parts—1 per cent for stone, 9 per cent for sand, and 15 per cent for silt—are the natural moisture for the stone and the optimum for sand and silt.

For the stone and sand only,

$$\text{voids in 1 cubic foot of stone} = 1 - \frac{103}{2{\cdot}52 \times 62{\cdot}4}$$
$$= 0{\cdot}34 \text{ cubic foot, or 34 per cent.}$$

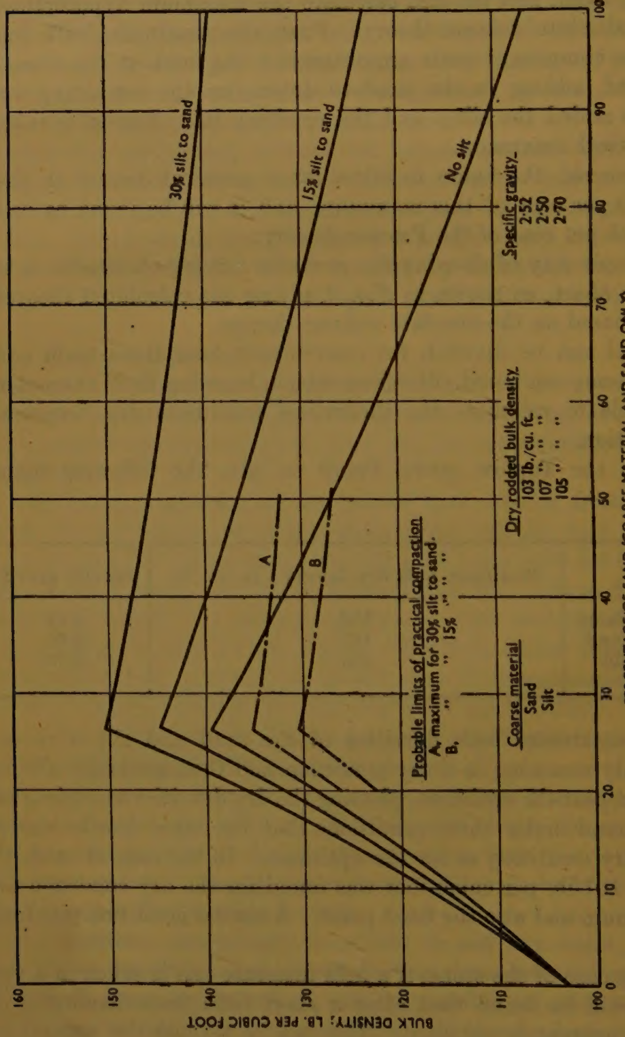


Fig. 2

COMPACTION OF GRAVEL—BULK-DENSITY CURVES

The quantity of sand required to fill these voids is 34 per cent of 107 lb., which equals 36·4 lb. The total weight of the combination is 103 + 36·4 = 139·4 lb. per cubic foot, and the percentage of sand to the total is 26·1 per cent. This percentage and density gives the peak on the "no silt" curve in *Fig. 2*.

$$\text{Voids in 1 cubic foot of sand} = 1 - \frac{107}{2{\cdot}6 \times 62{\cdot}4}$$

$$= 0{\cdot}34 \text{ cubic foot, or 34 per cent.}$$

Thus in 1 cubic foot of coarse and sand combined there is 34 per cent of 34 per cent = 11·6 per cent of voids.

The maximum quantity of silt to fill the voids is therefore equal to 11·6 per cent of 105 lb. = 12·2 lb., and the total weight of the combination is 103 + 36·4 + 12·2 = 151·6 lb. for 1 cubic foot. The maximum percentage of silt is equal to 8·05 per cent.

In the curves shown in *Fig. 2* the bulk densities are given for 15 per cent silt to sand and 30 per cent silt to sand, which are roughly the limits of the type of material found at the Airport, and the maximum percentage of silt to sand is equal to 33·6 per cent.

Determination of Points on the Coarse and Sand Curve only

(*a*) Below the maximum of 26·1 per cent.

Let W_c denote weight of coarse material
W_s „ „ sand
W_g „ „ combined material or gravel

Up to 26·1 per cent of sand W_c is always 103 lb. per cubic foot because the voids have not been filled.

$$\text{Then} \qquad \frac{W_s \times 100}{103 + W_s} = \text{percentage of sand.}$$

For example, 5 per cent of sand gives W_s as 5·43 lb. and the weight per cubic foot of the combination is 103 + 5·43 = 108·43 lb.

Other points in the curve up to 26·1 per cent sand are determined in the same way.

(*b*) Above 26·1 per cent sand

$$\text{Percentage of sand} = \frac{100W_s}{W_s + W_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1),$$

but in this case more than 1 cubic foot of combined material is produced

$$\text{and the volume} = 1 + \frac{W_s - 36{\cdot}4}{107},$$

$$\text{therefore the density per cubic foot} = \frac{W_c + W_s}{1 + \dfrac{W_s - 36{\cdot}4}{107}} \quad . \quad . \quad . \quad . \quad (2)$$

For example, 30 per cent sand substituted in equation (1) gives W_s

as 44·1 lb., which in turn substituted in equation (2) gives 137·2 lb. per cubic foot.

Determination of Points on the Coarse, Sand, and Silt Curve

(*a*) Below 26·1 per cent sand and for 15 per cent silt to sand.

Let W_g denote weight of coarse and sand per cubic foot
W_s ,, ,, ,, sand in 1 cubic foot
p ,, percentage of silt to sand
W ,, weight per cubic foot of the combination

For example, it was found previously for 5 per cent sand that

$$W_g = 108{\cdot}43 \text{ lb. per cubic foot,}$$
$$W_s = 5{\cdot}43 \text{ lb. per cubic foot, and}$$
$$W = 108{\cdot}43 + 0{\cdot}15 \times 5{\cdot}43 \text{ per cubic foot,}$$

from which $W = 109{\cdot}23$ lb. per cubic foot and other points in the curve up to 26·1 per cent sand are calculated in the same way.

(*b*) Above 26·1 per cent sand.

Again, in this case it is necessary to take into account the increase in volume, and

$$W = W_g + p\,(p_1 W_g) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3),$$

where p_1 is the percentage of sand.
So that for 30 per cent sand, $W_g = 137{\cdot}2$ lb. per cubic foot, as before, and $W = 143{\cdot}37$ lb. per cubic foot. All other points in the curve for 15 per cent silt to sand are calculated in the same way.

The curve for 30 per cent silt to sand is drawn by substituting 30 per cent for p in equation (3).

A simpler type of chart is the triaxial shown in *Fig. 3*. This is the ordinary chart used for sieve analysis but with densities and optimum moistures superimposed. Thus, when sampling, all that is necessary is to sieve out the material into coarse, sand, and silt, which will be represented by a point on the chart. This point gives the maximum density and optimum moisture immediately. The probable density obtainable in the field would be about 90 per cent of the maximum.

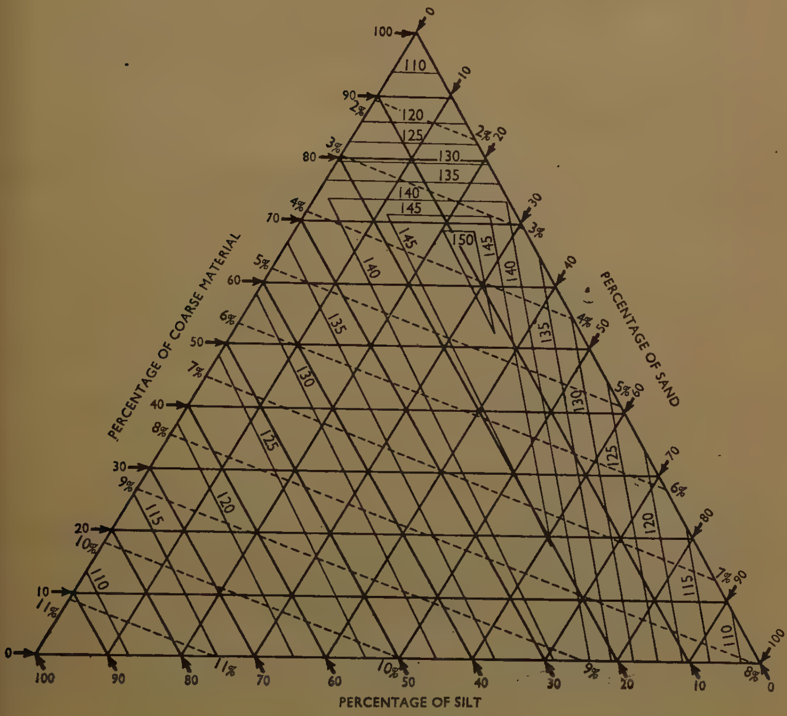
The use of this chart saved considerable time in the field. If, for example, difficulty was found in obtaining the minimum density in the field, a sieve analysis was taken and the probable density ascertained together with the correct optimum. It was frequently found in these cases that the sand percentage was over 40 per cent and the silt content was low, since the material from the large borrow pits sometimes varied to a considerable extent.

Methods Adopted for Density Testing

The standard method of making a density test is by digging a small hole, about 6 inches in diameter and 6 inches deep, and filling it with standard Leighton Buzzard sand with the aid of a sand bottle. The gravel taken from the hole is weighed and dried in a slow oven. The volume of the hole is computed by knowing the bulk density of the standard sand and the weight poured into the hole. From the above the dry density is found.

These operations should be repeated at least ten times to find an average for any one position, but it was necessary to reduce them to simple routine methods to cope with the large amount of gravel compacted each day.

Fig. 3



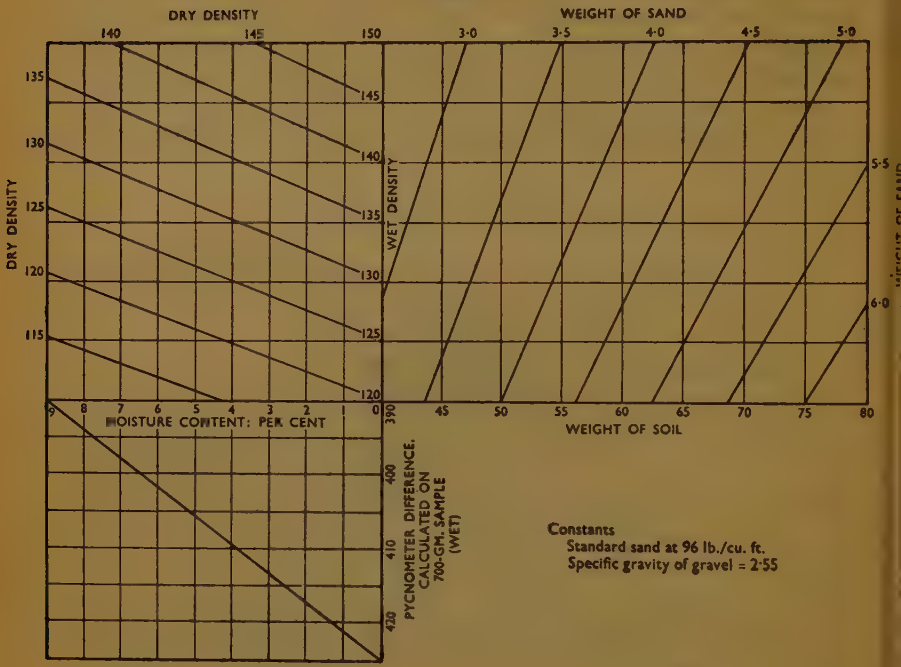
Compaction of Gravel—Triaxial Chart showing Maximum Densities and Optimum Moisture

The apparatus used on site was a large screw driver for digging and loosening the gravel and a large spoon for collecting it.

The standard sand was kept in canisters of 4 inches diameter and 15 inches long, with a $\frac{3}{8}$-inch-diameter hole in the lid. The weighing scales used were of the ordinary commercial type and weighed to within $\frac{1}{4}$ ounce.

By drawing up a chart as shown in *Fig. 4*, a labourer was trained to use it and was able to carry out as many as forty complete tests in a day.

Fig. 4



Constants
Standard sand at 96 lb./cu. ft.
Specific gravity of gravel = 2·55

Compaction of Gravel—Chart for Calculating Density

A small van was fitted with scales, sand canisters, and water for the pycnometer which was used for determining the moisture content.

The labourer dug a hole, carefully collecting the material taken from it; this was weighed and the hole filled with sand from the canister, the rate of flow of the sand being controlled with a finger placed over the hole in the lid. The canister being weighed before and after use gave the quantity of sand used. Then by finding the moisture content of the gravel, using the pycnometer, the wet density and dry density can be traced round the chart.

It will be noted that the pycnometer graph is based on pycnometer differences; this is desirable because pycnometers are frequently broken as they consist of 2-lb. fruit bottling jars with a metal cone substituted for the usual glass cap.

Pycnometer differences are determined as follows:—

Let w denote weight of surface dry gravel
p ,, ,, ,, pycnometer filled with water
P ,, ,, ,, pycnometer filled with water and gravel of weight w
S.G. ,, specific gravity of the gravel

Then
$$P = p + w - \frac{w}{\text{S.G.}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and
$$\text{S.G.} = \frac{w}{p + w - P} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

To determine the moisture content,

let W denote weight of wet sample
x ,, weight of surface moisture
$W - x$,, w = weight of surface dry sample

and P, p, and S.G. be as before.

Then from equation (1),

$$P = p + (W - x) - \frac{W - x}{\text{S.G.}}$$

and
$$x = W - \frac{(P - p)\,\text{S.G.}}{\text{S.G.} - 1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

In practice it is usual to express the moisture content as a percentage of the dry weight of the sample.

Therefore surface moisture as a percentage of dry weight $= \dfrac{100x}{W - x}$;

by interpolating for x,

$$\text{percentage surface moisture} = \frac{100W\,(\text{S.G.} - 1)}{(P - p)\,.\,\text{S.G.}} - 100 \quad . \quad . \quad . \quad (4)$$

Equation (4) is now reduced to a useful form since, for any material of known specific gravity, it is only necessary to keep a constant weight for the sample and, since " p " is constant for any one pycnometer, the equation becomes

$$\text{moisture as a percentage of dry weight} = \frac{C}{P - p} - 100,$$

$$\text{where } C = \frac{100W\,(\text{S.G.} - 1)}{\text{S.G.}}.$$

Thus, when making a moisture test, the operator has only to find the weight " P " for a constant weight W (700 grammes is about the right quantity for a 2-lb. fruit bottling jar), from which is subtracted the constant weight " p " and the percentage moisture found. A graph, as shown, can be rapidly drawn and need not be destroyed should the pycnometer be broken.

General

The dry density of the natural gravel immediately beneath the brick earth was 126 lb. per cubic foot and the average dry density of the compacted gravel (from 10,000 tests) was 133 lb. per cubic foot.

The contractor was given a minimum figure of 128 lb. per cubic foot and the results ranged from that figure to 138 lb. per cubic foot.

The question of setting out a bill of quantities for this class of work is rather important because, from a borrow pit of, say, 100,000 cubic yards measured in the solid state, about 95,000 cubic yards of compacted gravel would be obtained, and the amount transported would be of the order of 115,000 cubic yards.

If the contractor is to be paid only for the gravel measured in the solid, either he should be given the opportunity of ascertaining the percentage compaction and bulking, or basic figures should be given in the bill to keep all tenders on a competitive basis, otherwise excessive prices will be given to cover for these unknown factors.

The degree of accuracy of testing by the methods described, with ordinary care, is quite near enough for field purposes.

The commercial scales used for weighing might lead to small errors but the error is not likely to be more than ⅛ ounce, and even if the soil is weighed ⅛ ounce high and the sand ⅛ ounce low the error in computing the density is less than 1 lb. per cubic foot.

When using the pycnometer, however, scales reading to 1 gramme should be used, otherwise very large errors occur, as can be seen from the graph in *Fig. 4*.

If exceptionally high results are obtained, this is usually because the moisture content is over optimum and there is a slight collapse of the hole. Low results are obtained when the gravel is over-rolled in a dry condition. The larger stones then occupy the top 2 or 3 inches and will give a lower density.

A shallow hole can also give low results for the same reason, and the weight taken from a hole should not be less than 4 lb., with a diameter of about 4–5 inches.

A preponderance of clay or silt will give low results as can be seen from the tri-axial chart in *Fig. 3*.

Gravel should be spread in layers of even thickness to obtain consistent results. Rolling must be carried out at the optimum moisture necessary for the type of roller used. A sheepsfoot roller with tapered feet would compact about 9 inches, a rubber-tired roller about 6 inches, and a power roller of 8–10 tons about 3–4 inches.

If the gravel had a high percentage of clay, as distinct from fine silt, sheepsfoot rolling was found to be best, but should this type of material be rolled with a rubber-tired roller, difficulty would be experienced if it were over optimum. Below optimum moisture, the rubber-tired roller would give low results and above optimum the water would be trapped below the surface and a spongy condition produced, which is difficult to eliminate and it is usually best to have it removed.

When the gravel had a high percentage of silt, the rubber-tired roller was the most efficient machine. The power roller could never be used for general compaction because the layers had to be too thin to be economical.

In general, it was preferable to roll at under optimum whatever roller was used and when it became necessary to roll at over optimum the rubber-tired roller would pick out the wet spongy spots which could be treated separately.

When the gravel was fully compacted—that is, to air voids of about 10 or 11 per cent—rain did not affect it and only the top 1 inch, at the most, was affected by frost.

General traffic should be kept off the formation, even when fully compacted, because in the summer months traffic loosens the top and the fines are blown away whilst in the winter the fines work to the top and remain as a slurry.

In either case the top 1 inch or so would require further treatment before concreting but the formation below would be unaffected.

If during the winter months borrow material can be obtained at or below optimum, no difficulty would be experienced in compacting since about $\frac{1}{5}$ inch of rain is equivalent to a 1-per-cent rise in moisture content on a 9-inch layer; only exceptional rain would hold up operations.

CONCRETE

Introduction

The problem of the variability of concrete test cubes has exercised the minds of engineers for some considerable time and has led to many suggestions as to the causes.

Excessive values have been put on such things as grading curves, water/cement ratio, and shape and size of aggregate, but when put into their proper perspective in the design, mixing and placing of concrete assume values commensurate with their proper functions.

Much confusion of thought has arisen from the conversion of the old methods of specifying concrete mixes of the $2:1:n$ type to that of aggregate/cement ratios by weight.

The assumption that sand of one-third of the aggregate volume was suitable for all mixes is now known to be quite wrong. In the old volumetric theory it was known that one-third of sand would fill the voids in the coarse aggregate but, in an attempt to obtain the desired degree of workability, water was added. It can be shown that workability is not controlled by the addition of water but by the percentage of matrix in a unit volume.

This becomes obvious when a unit volume of coarse aggregate is considered, compacted to its maximum density. The voids in this volume can be filled with a matrix of any desired proportions, within certain limits, but it would not be possible to work the mix in any way. Whether a very rich matrix or not were used, workability would not be affected even if there were a considerable quantity of free water. On the other hand, if the unit volume were partially filled with coarse aggregate and the voids and remainder of the volume filled with the matrix, the effect would be to keep the coarse particles apart and thus promote ease of working.

The degree of workability can therefore be expressed as the quantity of matrix in a unit volume. Since this matrix, or mortar, is composed of three component parts which can vary one with the other, it is simpler to confine all calculations to the amount of coarse aggregate in a unit volume.

With this definition of workability the problem of grading curves is largely eliminated, because the percentage of sand would be decided by the degree of workability required.

Workability is also affected by the size and shape of the aggregate, since larger stones are more easily worked than smaller ones and round ones more than angular ones.

It therefore becomes possible to design a suitable mix, on paper, once the type of material to be used is analysed.

There is never any need to import to any local aggregate stones of different shape or size, or sand of a different fineness, because any desired degree of workability can always be obtained with the local product.

From the above statements it was possible, on the Airport contracts, to modify the volumetric mixes specified to suit the methods of placing the concrete by the various contractors. These modifications were accomplished without altering the cost of the materials by any appreciable amount, for a cubic yard of concrete.

One of the principal modifications carried out was in a 7½ : 1 concrete, by weight, with a water/cement ratio of about 0·55, and although mixing was by volume in a continuous mixer the results obtained were comparable with the high-grade 6·2 : 1 concrete used for the runways, which was batched by weight under very strict control.

From other results from different concretes and from a few experiments it was found that for the same water/cement ratio, within certain limits, the leaner concrete was stronger than the richer one, and in investigating this problem it was discovered that there is an optimum water content for a mix similar to that of soils. The strength of concrete appears to depend upon this optimum water content, and Abram's formula applies only under these conditions with full compaction.

From this it is obvious that the strength depends upon the amount of water and air voids in the concrete, and a different formula is required for strength of concrete having an excess of free water—that is, when a slump is used.

Full compaction is a term frequently used but the problem of obtaining it is seldom mentioned. It is logical to assume that full compaction can be obtained only at an optimum moisture content in the same way as soil compaction. If a voids curve is drawn for a concrete mix, it is possible to determine a curve of compaction which is similar in shape to that of a soil.

When the conditions are over or under optimum it is impossible to reach full compaction or full plastic density.

The principle adopted by the Author in determining the optimum moisture for gravel was used on concrete mixes, and in the case of mixes with a maximum size of 1½ inch the optimum moisture was 1 per cent for the coarse, 9 per cent for the sand, and 24 per cent for the cement, in each case, by weight.

For less water than the above it appears impossible to apply sufficient vibration, or force, to the mix to produce full hydration of the cement, and when more than the minimum of water is used it is difficult to expel all the trapped air. In both cases full compaction is not achieved with a consequent loss in strength.

Before giving details of results obtained at London Airport, it is necessary to explain the theory developed and the methods used, together with experiments carried out in a field laboratory with the minimum of equipment.

Minimum Water Required for a Mix

In considering the aggregate for a concrete mix, it is only necessary to separate the coarse from the fine material.

The division of coarse and fine material is made on the $\frac{3}{16}$-inch sieve and the material passing this sieve will show considerable bulking, from the dry condition to a maximum, at a little over the halfway line between the dry and flood points.

Material larger than $\frac{3}{16}$ inch does not show any bulking—at least, none that can be measured by ordinary means.

The determination of the optimum moisture content for the sand was found by the usual Proctor density method, and for the sands used on site ranged from 8–10 per cent.

The alluvial sands, which were the coarsest, had a fineness modulus of about 265 to 270, whilst the flood plain sands were as high as 350. The sands were mixed in the mixer bins producing a combined modulus which varied between 300 to 320, the optimum of which was about 9 per cent.

With the material larger than $\frac{3}{16}$ inch in size, the Proctor density test cannot be applied. A free particle of aggregate will carry a certain amount of water due to the capillary action and surface tension of the water. A collection of particles ultimately carry only this amount of water, which can be referred to as the natural moisture content.

The thickness of the natural water film on stone is probably of the order of 0·003 inch. This can be determined by collecting well-rounded stones and testing for the thickness of the film, or by selecting individual stones and checking on micro-scales.

With the collection of stones, as near as possible all of the same size, the test is carried out by making a heap and flooding with water. Moisture tests are taken at intervals and the results plotted. The curve produced will be hyperbolic in shape for a time and then will become a straight line with a marked slope not tangential to the curve. The hyperbola indicates drainage and the straight line evaporation.

If the test is carried out in a large stock pile, evaporation is extremely small compared with drainage, and it is found that after a period of a few hours the moisture content is constant, indicating the lower point of the hyperbola.

With $1\frac{1}{2}$-inch to $\frac{3}{16}$-inch material this figure will be approximately 1 per cent, and with the 9 per cent for the sand the amount of water is the minimum which can be used to make concrete with the addition of the necessary hydration water of the cement.

If it is accepted that the thickness of the water film is 0·003 inch, then it is possible to determine by calculation the natural moisture content of any graded coarse aggregate.

Considering spheres of stone of the same surface texture and specific gravity, the weight of the water film is given by $\pi d^2 tr$, where d denotes the diameter of the sphere, t the thickness of the water film, and r the density of water.

The weight of the sphere is $\frac{\pi}{6} d^3 s$, where s denotes the density of stone.

Therefore the percentage moisture content $= \dfrac{\pi d^2 tr 100}{\frac{\pi}{6} d^3 s}$; the quantity $\dfrac{tr100}{s}$ is a constant, say K, so that the percentage moisture content is $\dfrac{6K}{d}$.

If a cube is considered then the result is again $\dfrac{6K}{d}$, where d is equal to the side of the cube.

For any particle whose dimensions are x, y, and z, where x is the least dimension,

let $y = ax$ and $z = bx$,

then the surface area $= 2\,(xy + xz + yz)$

$= 2x^2(a + ab + b)$.

Volume $= xyz = abx.^3$

Omitting the constant " K ",

$$\frac{\text{area}}{\text{volume}} = \frac{2x^2(a + ab + b)}{abx^3}$$

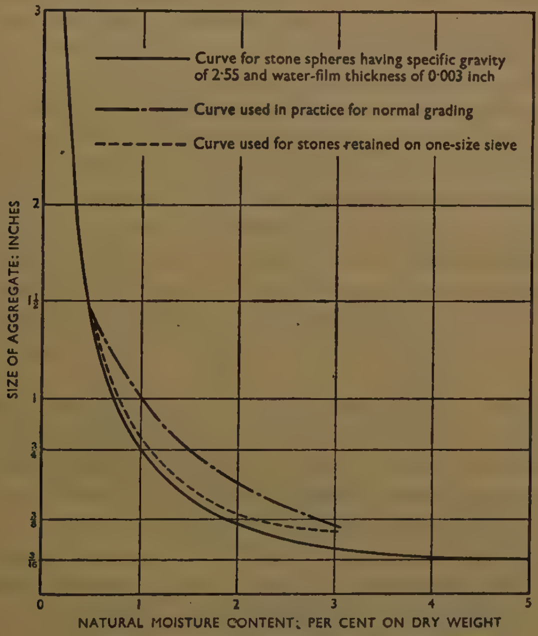
$$= \frac{2}{x}\left(1 + \frac{1}{a} + \frac{1}{b}\right).$$

For a lamina where a and b are large compared with x, the ratio of area to volume tends to $2/x$, and where $a = b = 1$ the ratio is $6/x$.

Fig. 5 shows a curve of natural moisture content of stones which have a specific gravity of 2·55 and, in the case of spheres, a water film of 0·003-inch thickness.

As can be seen, the curve for stones retained on one sized sieve follows closely the curve for spheres. The curve for nominal grading leaves the spheres curve because in nominal grading the stones are not all of one size and it is usual to allow a small percentage to pass the sieve of the next smaller size. The reason that both curves tend to leave the spheres curve at about the ⅜-inch size is that the capillary action which takes place allows the water to fill the interstices between the very small particles.

Fig. 5



NATURAL MOISTURE IN COARSE AGGREGATE

To ascertain the natural moisture in 1½-inch to $\frac{3}{16}$-inch aggregate, the sieve analysis should be obtained and, by proportion from the spheres curve, the total found.

For example, assuming 50 per cent of 1½ inch, 30 per cent of ¾ inch, 15 per cent of ⅜ inch, and 5 per cent passing the ⅜-inch sieve, the corresponding percentage moistures are 0·25 per cent for 1½ inch, 0·30 per cent for ¾ inch, 0·30 per cent for ⅜ inch, and 0·15 per cent for less than ⅜ inch—a total of 1 per cent. It can be seen that variations in the sieve analysis have little effect upon the total. Similar calculations can be made for any grading curve for stones.

The shape factor has very little effect upon coarse aggregates unless they are laminated like crushed flints.

The area to volume ratio of $6/d$ should be increased to about $5/d$ for crushed flints, but there is no need to make any change for any other aggregates used for concrete, apart from special types like foam slag or other porous aggregates. For these special cases the natural moisture curve should be determined by experiment, with due allowance for absorption.

Having indicated a method of finding the minimum amount of water required for the aggregates, the next problem is to determine the hydration point of the cement.

The 24 per cent used by the Author was obtained from numerous tests, but it should be pointed out that this is not necessarily the true hydration point and it is probably better to refer to it as the apparent hydration.

The following test example will show why this is so.

The mix was one of the standard Air Ministry mixes, 7·7 : 1 by weight, using a maximum size of aggregate of 1½ inch. The specific gravity of the stone was 2·54 and the sand 2·60, whilst the cement was always taken as 3·15. The mix used for the test was 18·3 lb. of stone, 12·25 lb. of sand, 3·975 lb. of cement, and 2·385 lb. of water.

From this the calculated plastic density was 148·1 lb. per cubic foot (the weights per cubic foot being 73·5 lb. for stone, 49·2 lb. for sand, 15·9 lb. for cement, and 9·5 lb. for water), and, assuming 24-per-cent hydration, the free water was 5·7 lb.; the dry density was therefore 142·4 lb. per cubic foot.

When this free water is dried out air voids remain, and in this case was 9·1 per cent by calculation. The actual results obtained were 147·58 lb. per cubic foot plastic density, and 141·77 lb. per cubic foot dry density, after drying out to constant weight. The loss of water was 5·81 lb. in 1 cubic foot of concrete, which gave natural voids of 9·35 per cent, but as full compaction was not reached by 0·35 per cent the total voids were 9·7 per cent. The total water for 1 cubic foot of concrete was 9·5 lb., so that hydration water was 9·5 − 5·81 = 3·69 lb. This 3·69 lb. of water used for the hydration of the cement was 23·2 per cent of the weight of cement in 1 cubic foot.

The above is an approximate calculation and it should be corrected for the degree of compaction, which was less than 100 per cent, as can be seen from the difference in the percentage of calculated and actual air voids (*Fig. 7*).

The result, however, is accurate enough to show the method and a check can be kept on the hydration of the cement used during a contract by occasionally taking cubes for density tests.

Tests on a very large number of cubes at London Airport have given results ranging from 23 to 25 per cent.

Density and Compaction

The problem of air voids and density is one which could do with a little more consideration. The plastic density of a mix can be determined in a few minutes whereas a strength test usually takes 7 days.

The measure of density depends upon the air voids in the concrete, and the air voids are caused by the following factors :—

(1) The matrix of cement and sand mortar is not sufficient to fill the voids in the stone.
(2) Honeycombing caused by incomplete compaction.
(3) Voids caused by the "free" water drying out.

Causes (1) and (2) are avoidable but (3) cannot be avoided ; it can, however, be reduced to a minimum.

In case (1) the condition of not filling the voids is met with only in very lean mixes, whilst honeycombing can be avoided by ensuring that the required workability is obtained for the type of vibration applied.

In case (3) it is not difficult to calculate the voids for any individual mix, but the following method enables a composite graph to be drawn, showing the variation in air voids plotted against aggregate/cement ratios.

Adopting the following notation :—

A denotes aggregate in lb. (S.G. $= 2{\cdot}55$) W denotes total water in lb.
C ,, cement in lb. (S.G. $= 3{\cdot}15$) R ,, aggregate/cement ratio
r ,, water/cement ratio

Hydration of cement = 24 per cent by weight,

then absolute unit volume $= \dfrac{A}{2{\cdot}55} + \dfrac{C}{3{\cdot}15} + \dfrac{W}{1}$,

putting C and W in terms of A,

$$= \frac{A}{2{\cdot}55} + \frac{A}{3{\cdot}15R} + \frac{rA}{R}$$

$$= \frac{A}{R}\left(\frac{R}{2{\cdot}55} + \frac{1}{3{\cdot}15} + \frac{r}{1}\right) \quad . \; . \; . \quad (1)$$

The absolute volume of free water $= \dfrac{rA}{R} - \dfrac{0{\cdot}24A}{R}$

$$= \frac{A}{R}(r - 0{\cdot}24) \quad . \; . \; . \; . \; . \; . \quad (2)$$

Therefore percentage of air voids $= \dfrac{\dfrac{A}{R}(r - 0{\cdot}24) \times 100}{\dfrac{A}{R}\left(\dfrac{R}{2{\cdot}55} + \dfrac{1}{3{\cdot}15} + \dfrac{r}{1}\right)}$

$$= \frac{100r - 24}{0{\cdot}3922R + 0{\cdot}3175 +} \quad . \; . \; . \quad (3)$$

From equation (3) a composite graph (*Fig. 6*) can be drawn. An inspection of this graph shows that for the same water/cement ratio the leaner mixes are denser.

For an individual mix a density chart can be drawn to show how honeycombing occurs because of insufficient compaction.

Fig. 6

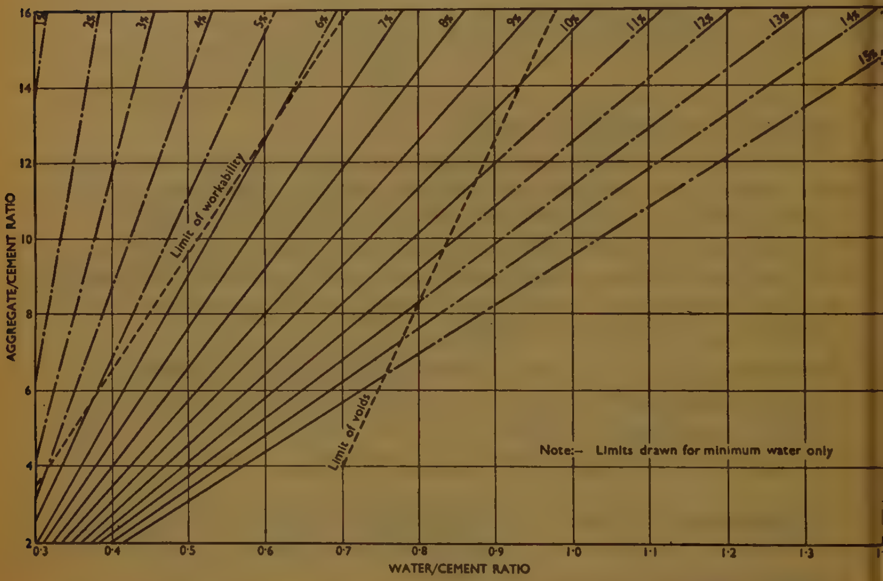


DIAGRAM OF AIR VOIDS IN FULLY-COMPACTED CONCRETE AFTER EVAPORATION OF FREE WATER

Considering the mix of 7·7 : 1 on which the hydration test was made, the density chart appears as shown in *Fig. 7*.

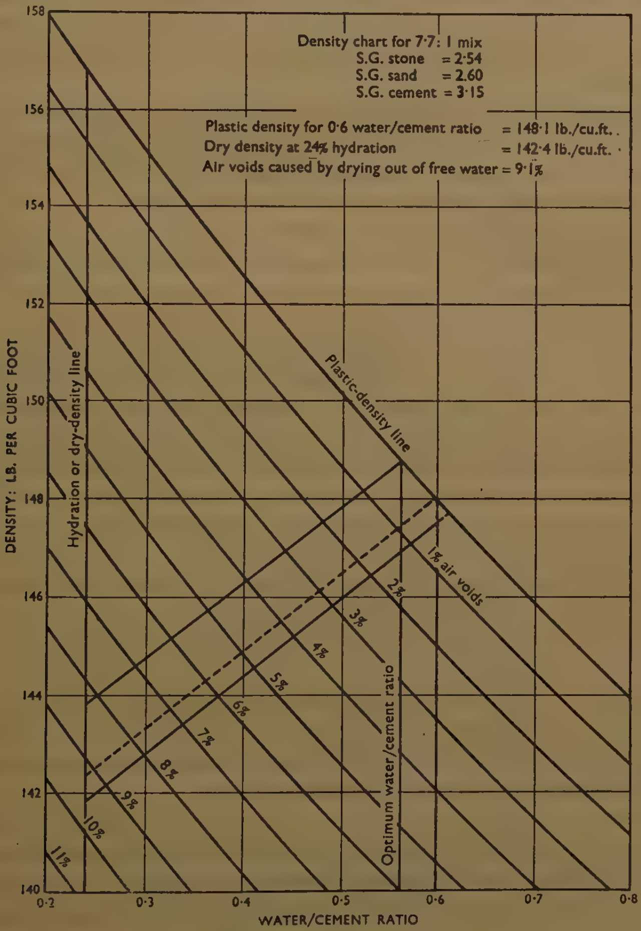
For the water/cement ratio of 0·6 the plastic density is shown to be 148·1 lb. per cubic foot and, by drawing a diagonal line from this density to the dry density of 142·4 lb. per cubic foot where it intersects on the hydration line of 24 per cent, the percentage of air voids is shown to be 9·1 per cent, as previously calculated.

This chart is similar to the one used in soil compaction and there is obviously a point of optimum moisture where the maximum plastic density can be reached. It is probable that this occurs when minimum water is used to lubricate the aggregate and hydrate the cement.

For the mix in question, basing the minimum water on 1 per cent for the 1½-inch stone, 9 per cent for the sand, and 24 per cent for the cement, the water/cement ratio is 0·563.

The plastic density in this case is 148·8 lb. per cubic foot and is the maximum which can be attained, whilst the dry density would be 143·8 lb. per cubic foot and air voids 8·2 per cent.

Fig. 7



Density Chart

For a water/cement ratio of lower than 0·563 it would be impossible to reach the plastic density line. The density obtained would not reach the diagonal line joining the plastic density of 148·4 lb. per cubic foot to the dry density of 143·8 lb. per cubic foot.

When the water/cement ratio is higher than 0·563 it again appears impossible to reach the plastic density line, although coming very near to it.

Abram's formula for strength presupposes 100-per-cent compaction, but it is felt that a further qualification should be made in view of the foregoing, as it appears that the formula applies only at optimum water/cement ratio.

When studying the effects of compaction and knowing that the strength fell off rapidly with very small percentage reductions, it was thought that the strength was proportional to the amount of voids and correspondingly the density.

An examination of the density chart shows that for the optimum water/cement ratio, if the mix is not compacted to the plastic density line, then the additional voids must be added to the free-water voids, which remain constant for the water/cement ratio used.

It was, therefore, reasonable to assume that the extra voids could be treated as additional free water and a new equivalent water/cement ratio found. On this assumption a curve such as that shown in *Fig. 8*, drawn for the 7·7 : 1 mix, can be constructed.

For its construction, lines are drawn parallel to the line joining the optimum plastic density to the dry density on the hydration line, through successive air voids lines where they intersect with the vertical through the optimum water/cement ratio. Each of these lines is continued through to the plastic density line and the increased water/cement ratio read off and the corresponding strength calculated. The plastic or wet densities are read off where the vertical optimum water/cement ratio cuts each of the voids lines.

For the curve shown the Author used a straight-line formula for strength, which was found to be well within the capabilities of the average contractor:

$$\text{strength at 7 days in lb. per square inch} = 2{,}800\left(\frac{c}{w} - 0{\cdot}5\right).$$

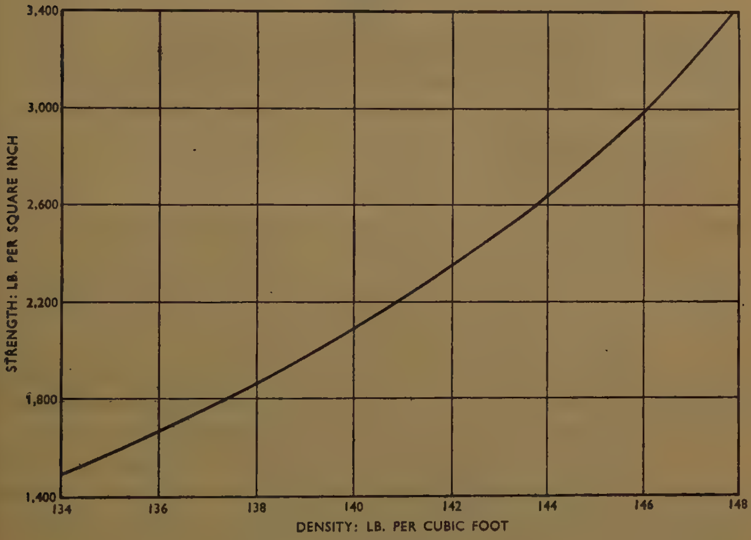
This formula will be found to give slightly higher results than the curves usually published and based on Abram's formula.

To obtain proof of the type of curve shown in *Fig. 8* a number of cubes were tested for density and strength, with the results shown in *Fig. 9*. It will be noticed how the results are scattered above and below the line. At each density an average was taken of the strengths and when plotted fell very closely to the line, indicating that the water/cement ratio was fairly well maintained and that the voids or density is proportional to the strength.

The large variations above and below the line show how the strength of the cement varied. It was unfortunate that, at the time of doing this test, no check was made on the strength of the cement, but its variability has been checked from time to time on the contract and nothing out of the way was indicated in the results obtained.

A further point to note is that density/strength curves vary for different mixes. This view is borne out by the statements made by M. Freyssinet,[1] when he suggested that Abram might be wrong and that strength was probably proportional to the voids/cement ratio. Further investigation is necessary on this subject but it was beyond the scope of the Author whose field laboratory is intended for constructional work and not organized research.

Fig. 8



Strength/Density Curve
(7·7 : 1 mix, 0·563 water/cement ratio (optimum))

Workability

Workability can be defined, as previously stated, as the amount of coarse aggregate in a unit volume of concrete. This quantity will vary with the shape, size, and grading of the coarse aggregate, and the fineness modulus of the sand.

Considering the shape of the coarse aggregate, it is obvious that well-rounded stones are more workable than angular or irregular shapes, and for the same specific gravity a greater weight of rounded stones can be used.

Size and grading are interdependent. Small stones are more difficult to work than large ones so that, for the same workability, a greater weight of larger stones can be used.

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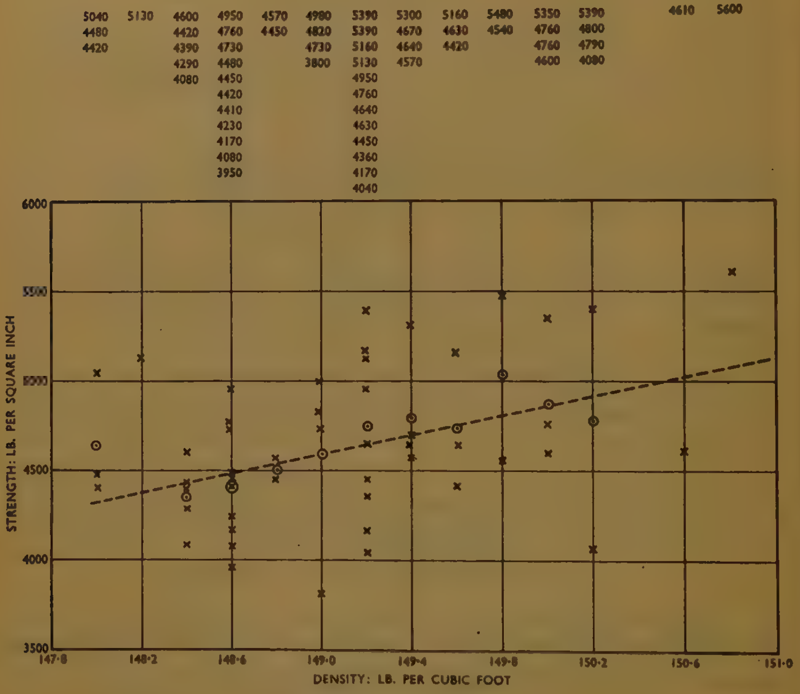
[1] E. Freyssinet, "Pre-stressed Concrete: Principles and Applications." J. Instn Civ. Engrs, vol. 33, p. 331 (November 1949).

Variation in the grading of the sand has a small effect on the workability of a mix, but it is very noticeable when using a surface vibrator on paving slabs. For the same degree of workability less fine than coarse sand is required, and for a difference of about 100 in the fineness modulus there would be a 2-per-cent variation in the quantities required.

Fig 9.



DENSITY/STRENGTH CURVE

(6·2 : 1 mix, 36 per cent sand, 0·48 water/cement ratio)

Correct workability is an important factor in the design of a concrete mix.

At London Airport an American type of surface vibrator was used, operating on the out-of-balance weight system which provides no amplitude in the beam and was suitable only for mixes having medium workability.

For the paving quality concrete, which was 6·2 : 1 aggregate/cement ratio by weight, the quantity of 1½-inch to $\frac{3}{16}$-inch coarse aggregate was 2,080 lb. per cubic yard of concrete.

The vibrator would not operate satisfactorily with a greater weight than 2,080 lb., but in passing it should be mentioned that there are British-made machines which would operate at a much higher figure with all the advantages of being able to use a smaller percentage of sand.

The contractors, having only this type of American vibrator on the site, wished to use it for other mixes and in some instances those required modification to enable them to be vibrated to full compaction.

The under slab used for the main runways and tracks was specified as 12 : 1 concrete, or 12 cubic feet of 1½-inch down all-in ballast to 1 cwt. of cement. The natural sand content of the ballast was between 35 and 37 per cent and the water/cement ratio used was about 1·1. The workability of this mix was therefore about 2,100 lb. per cubic yard of concrete and, although a little high for the vibrator, it was used and gave fairly good results.

The results obtained were about 2,000 lb. per square inch at 28 days with about 97-per-cent compaction, which was considered satisfactory for the under slab.

For roads and hangar floors the mix specified was 6½ cubic feet of 1½-inch to $\frac{3}{16}$-inch loose coarse aggregate, 3¼ cubic feet of loose moist sand to 1 cwt. of cement, with a slump of 2 inches and a strength of 2,000 lb. per square inch at 28 days.

The loose bulk densities of the coarse and fine aggregates were found under site conditions with the following results.

	Wet density : lb./cu. ft.	Moisture content : per cent	Dry density : lb./cu. ft.	Specific gravity
Coarse aggregate	97·41	0·4	97	2·54
Sand . . .	92·37	5·9	87·4	2·6

From these figures the yield of materials per cubic yard was computed and gave :—

Stone	2,270 lb.
Sand	980 lb.
Cement . . .	423 lb.
Water	278 lb.

The aggregate/cement ratio by weight was 7·7 : 1, sand 30·1 per cent, and water/cement ratio 0·66.

For the water/cement ratio used a slump of ¾ inch was obtained and any attempt to raise that to the 2 inches specified would have resulted in segregation.

As can be seen, the quantity of stone per cubic yard was high and would give low workability far outside the range of the vibrator used.

By drawing a yield chart (*Fig. 10*) for the 7·7 : 1 mix, it is possible to determine the best mix to use.

First, the minimum water line is calculated, basing the required water on 1 per cent for the stone, 9 per cent for the sand, and 24 per cent for the cement, and taking 770 lb. of aggregate to 100 lb. of cement.

With 0 per cent sand

Stone, 770 × 1 per cent = 7·70
Cement, 100 × 24 ,, ,, = 24·00

31·70 lb. of water

= 0·317 water/cement ratio.

With 50 per cent sand

Stone, 385 × 1 per cent = 3·85
Sand, 385 × 9 ,, ,, = 34·65
Cement, 100 × 24 ,, ,, = 24·00

62·50 lb. of water

= 0·625 water/cement ratio.

The straight line joining these two points is the minimum water line.

Next, the segregation line is calculated. This occurs at the flood point of the sand, and for the medium sand used on the site was equal to 17 per cent. With 0 per cent sand the water/cement ratio must be as for the minimum water condition.

For 50 per cent sand

Stone, 385 × 1 per cent = 3·85
Sand, 385 × 17 ,, ,, = 65·35
Cement, 100 × 24 ,, ,, = 24·00

93·20 lb. of water

= 0·932 water/cement ratio.

To calculate the workability lines, it is necessary to use the absolute volume equations.

The sum of the absolute volumes being equal to unity:

$$V_a + V_s + V_c + V_w = 1,$$

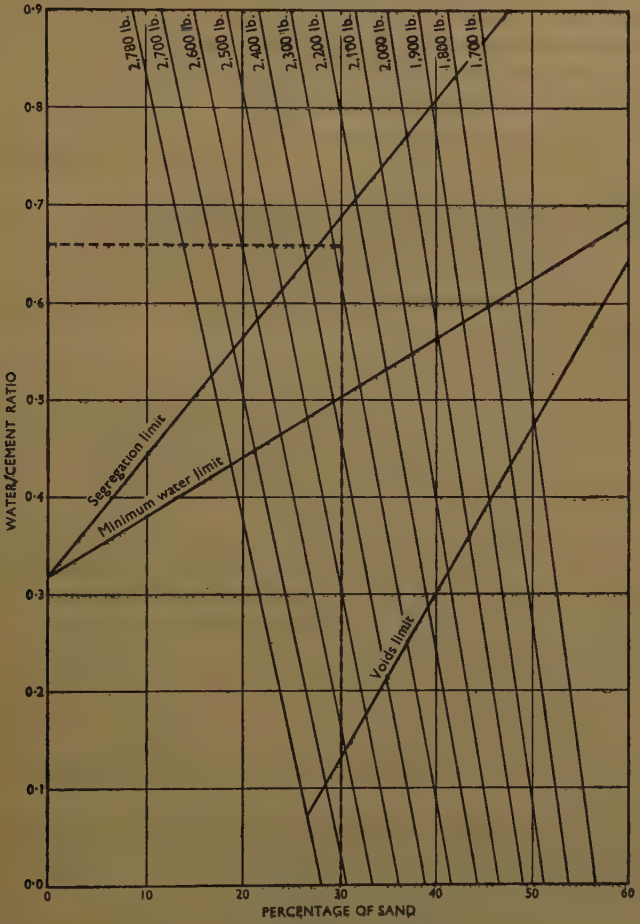
where V_a, V_s, V_c, and V_w are the absolute volumes of the coarse aggregate, sand, cement, and water respectively.

The equation can be converted to weights per cubic yard of concrete:

$$\frac{W_a}{S_a} + \frac{W_s}{S_s} + \frac{W_c}{S_c} + \frac{W_w}{1} = 1{,}684{\cdot}8 \text{ lb.},$$

where W denotes weight and S the specific gravity.

Fig. 10



YIELD CHART FOR MIX
6½ cubic feet of coarse aggregate
3¼ „ „ „ sand (moist)
1 cwt. of cement
Aggregate/cement ratio by weight = 7·7 : 1
Water/cement ratio = 0·66
Maximum size of aggregate = 1½ inch

In this particular case, where the aggregate/cement ratio is 7·7 : 1,

$$W_c = \frac{W_a + W_s}{7 \cdot 7},$$

and W_w can be written as $r . \frac{(W_a + W_s)}{7 \cdot 7}$,

where r = water/cement ratio.

To draw the maximum workability line, the rodded bulk density of the coarse aggregate should be found. In general, it was of the order of 103 lb. per cubic foot, or 2,780 lb. per cubic yard.

By selecting two water/cement ratio points, say 0·8 and 0·2, W_s can be found. Thus for 0·8 water/cement ratio

$$\frac{2{,}780}{2{\cdot}54}+\frac{W_s}{2{\cdot}6}+\frac{2{,}780+W_s}{7{\cdot}7\times 3{\cdot}15}+\frac{0{\cdot}8(2{,}780+W_s)}{7{\cdot}7}=1{,}684{\cdot}8,$$

from which W_s is found and thence the percentage of sand.

By substituting 0·2 water/cement ratio, another point is found on the 2,780-line and the two points are joined.

Other values of workability can be calculated in the same way and the chart completed.

The point to note in this type of chart is that concrete can only be made in the area bounded by the minimum water line, the segregation line, and the maximum workability line of 2,780 lb. The mix as specified, which is indicated by the intersection of the 0·66 water/cement ratio, 30·1 per cent sand, and workability of 2,270 lb., is shown to be impossible to place with the medium type of vibrator used on site.

Further water could not be added to improve workability because segregation would have been reached at 0·69 water/cement ratio with a workability of about 2,250 lb. By reducing the water/cement ratio to about 0·55 and increasing the percentage of sand to about 38, a suitable workability could be obtained.

Starting from this point a mix was worked out and, converting back to volumetric ratios as the contractor did not wish to weigh batch in this case, the nearest simple fractional parts of 5½ cubic feet of stone and 3½ cubic feet of sand was decided upon, which gave 7·5 : 1 aggregate/cement ratio by weight.

This mix was perfectly workable and gave results in which the highest number of groups of cubes produced 5,700 lb. per square inch at 28 days with a coefficient of variation of 23 per cent.

This figure was reasonable as the mix was produced from a continuous mixer with very little control, apart from the designed balanced mix. Experience with this type of mixer indicated that frequent checking of parts is required and, generally, results were better on Mondays after a week-end overhaul of the machine.

The principal variation occurred in the wearing down or clogging of the cement feed screw which, after a period of working, gradually reduced the quantity of cement fed to the mix.

The coefficient of variation of 23 per cent could, no doubt, have been reduced if a batching plant had been used, either volumetric or by weight. The coefficient of variation of the cubes from the main weigh-batching plant was between 11 and 12 per cent, with an average strength of 5,500 lb. per square inch at 28 days, which would have been equalled or exceeded with the 7·5 : 1 mix and is sufficient proof that the leaner mix with a higher water/cement ratio was as good as the 6·2 : 1 mix. The relative water/cement ratios were 0·55 for the 7·5 : 1 mix and 0·48 for the 6·2 : 1 mix.

As can be seen from *Fig. 6*, for the same water/cement ratio, density increases as the mixes become leaner and it is reasonable to assume that strength is proportional to density.

To test this assumption a series of tests was made :—

(1) With a constant water/cement ratio of 0·57, two mixes were made with 6 : 1 and 8 : 1 aggregate/cement ratios.
(2) Two mixes of 6 : 1 aggregate/cement ratio were made with different water/cement ratios.
(3) Two mixes of 6 : 1 and 8 : 1 aggregate/cement ratios were made, each having the same air voids.

The results of these tests are shown in Table 1.

In test (1) the 8 : 1 mix was stronger than the 6 : 1 mix, and as far as could be ascertained both mixes had maximum compaction. In test (2) the obvious result was obtained, since the lower water/cement ratio produced the higher strength.

In test (3) there was very little difference in the strengths, thus showing that the strength depends on the voids in the mix.

The Author is not prepared, at this stage, to offer a new formula for strength because he feels that further research is required on this problem and that it comes within the province of a research laboratory rather than that of a field laboratory.

General

The few examples given are not all that have been observed or tested. For instance, a mix can be designed to suit any required condition in the matter of a few minutes with the methods described. This can be carried out by a few simple calculations without recourse to making any trial mixes.

As an example, the lining of the banks of two river beds, which had to be diverted at the Airport, were to be carried out in concrete at a slope of about 40 degrees. The concrete had to be compacted to maximum density by vibration and the mix suggested was the "no slump" concrete, as used for the runways. This concrete failed to stand at the slope required after a few passes of the vibrator and tended to slump at the bottom.

The vibrator used was of the compressed-air type, fitted to a short screed, and was more vigorous than the out-of-balance weight surface vibrator used on the paving.

The result was only to be expected, because the vibrator was too vigorous for the medium workability of the concrete although the mix had no slump. The obvious solution was to have as low a workability figure as possible to suit the vibrator, and a new mix was designed. The workability was increased to 2,200 lb. and the aggregate/cement ratio to 5 : 1

TABLE 1

Mix	Calculated		Actual	Compaction: per cent	Strength at 7 days	Strength corrected for 100-per-cent compaction	Grouping
	Plastic density	Voids: per cent	Plastic density				
A/C ratio 6 : 1, 38 per cent sand; W/C ratio 0·57, workability 1,950 lb.	146·99	9·0	146·28	99·5	2,420	2,510	Group "A": different A/C ratios, same W/C ratios, different voids
	—	—	146·61	99·7	2,300	2,350	
A/C ratio 8 : 1, 38 per cent sand; W/C ratio 0·57, workability 2,090 lb.	149·53	8·27	146·08	97·7	2,490	3,070	
	—	—	146·00	97·6	2,610	3,280	
A/C ratio 6 : 1, 32 per cent sand; W/C ratio 0·47, workability 2,210 lb.	149·5	6·26	149·20	99·5	3,920	4,080	Group "B": same A/C ratios, different W/C ratios, different voids
	—	—	148·50	99·2	4,170	4,430	
A/C ratio 6 : 1, 42 per cent sand; W/C ratio 0·51, workability 1,860 lb.	148·83	7·2	148·2	99·58	3,640	3,760	
	—	—	147·0	98·77	3,740	4,130	
A/C ratio 6 : 1, 41 per cent sand; W/C ratio 0·523, workability 1,880 lb.	148·21	8·94	146·98	99·17	3,950	4,320	Group "C": different A/C ratios different W/C ratios, same voids
	—	—	146·92	99·13	4,100	4,500	
A/C ratio 8 : 1, 41 per cent sand; W/C ratio 0·6, workability 1,980 lb.	149·14	8·95	145·68	97·68	3,200	3,940	
	—	—	145·78	97·75	3,400	4,180	

with no slump. The mix gave perfectly satisfactory results using only coarse and fine aggregates.

It should be noted here that the Author, so far, has not yet met a surface vibrator which will handle more than 2,260 lb. of 1½-inch to $\frac{3}{16}$-inch stone in a cubic yard of concrete. Immersion vibrators will handle a much higher figure, but for large areas they are uneconomical.

The 7·5 : 1 mix, referred to previously, was also used for the flooring of hangars and this work was frequently carried out by building contractors, as distinct from civil engineering contractors, with little experience in the production of high-quality concrete.

Since the mix was balanced to suit the type of vibration used, the results obtained were reasonably good, and varied between 4,500 and 5,500 lb. per square inch in 28 days. Left to their own devices and to the mix originally specified, the contractors' results would, no doubt, have been around the 2,000 lb. per square inch asked for in the specification.

The mixers used were generally 10S and the materials were gauged by volume in two sizes, coarse and fine, in the usual way. Once the mixer driver was shown the correct quantity of water to add and the appearance of the mix, he kept it reasonably consistent.

Concrete can be pumped, in approximately the no-slump condition, when it has medium workability.

Sufficient water must be added to the minimum water to provide lubrication to the delivery pipes. This is a very small amount and can be calculated by adding a quantity of water represented by a film 0·003 inch thick on the lining of the pipes for each unit volume pumped.

Concrete of just below medium workability will not segregate when dropped from heights of up to 20 feet provided that minimum water is used.

Natural segregation, or bleeding as it is sometimes called, occurs when there is about 17 per cent free water in the sand, irrespective of the workability.

In general, not more than 9 per cent moisture for the sand should be allowed unless it is for reinforced concrete, when 10 per cent is sufficient, giving a slump of ½–¾ inch. These slumps have been used to a considerable extent on reinforced concrete on the site with entirely satisfactory results.

For reinforced-concrete work the coarse aggregate was ¾ inch to $\frac{3}{16}$ inch and medium workability was about 1,800 lb. per cubic yard of concrete.

The sieve analysis to give the workability figures quoted for the 1½-inch to $\frac{3}{16}$-inch material is 50 per cent passing 1½ inch, 30 per cent passing ¾ inch, and 20 per cent passing ⅜ inch; whilst for the ¾-inch to $\frac{3}{16}$-inch material it is 65 per cent passing ¾ inch and 35 per cent passing ⅜ inch.

For any variation in the sieve analysis the workability figures would have to be modified, up or down, depending upon whether the material was coarser or finer.

The problem of control of concrete really lies in correctly designing the mix. Direct supervision is unnecessary and expensive, if the mix is badly designed, and in weigh batching the final control of a mix is by eye owing to the inherent difficulty of trying to measure the moisture content of the aggregate.

Volumetric batching is more consistent both in strength and yield than weigh batching, and requires less direct supervision.

In one case of a runway 3,000 yards by 100 yards supervised by the Author and carried out by volumetric batching during the later stages of the war, the results were comparable with weigh batching as regards strength and gave a much closer yield.

The Author was informed that if he insisted on volumetric batching results would be of the order of 3,000 lb. per square inch at 28 days; the actual results were 7,500 lb. per square inch. The calculated yield was within 1 per cent of the actual measured concrete and the average density of the runway was equal to the average density of the cubes.

With weigh batching, on a large scale, it is difficult to get the yield correct to within 5 per cent. The reason appears to be in the control of the mixer.

With volumetric batching only water has to be varied, but in weigh batching weights should be altered as well. This is frequently overlooked in the rush of production, with a corresponding alteration to the yield.

If sands of different finenesses are used, which is unavoidable on a large contract, the causes of the varying appearance of a mix are not directly apparent. If working to a no-slump condition, no change appears if there is too little water—the sand being fine—but any increase caused by the sand being coarse is immediately seen and the water reduced, thus reducing the yield. Weigh batching requires a much more consistent grading curve than volumetric batching.

In general, when weigh batching, the aggregates should be split up into as many sizes as possible, but in volumetric batching it is only necessary to split them into coarse and fine, which means a considerable reduction in cost.

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The Paper is accompanied by ten sheets of drawings, from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX

Design of a Concrete Mix

The design of a concrete mix depends, in the first instance, on the strength required. The only other requirement to know or specify is the degree of workability.

If density is also another desirable quality, then *Fig. 6* will show the maximum density for the mix selected.

Notation

W_a, W_s, W_c, W_w denote weight per cubic yard of concrete of coarse, fine, cement, and water
S_a, S_s, S_c ,, specific gravity of coarse, fine, and cement
P_a, P_s, P_c ,, minimum water required for coarse, fine, and cement
R denotes aggregate/cement ratio by weight

Knowing the strength required, the cement/water ratio can be found from

$$S \text{ at 7 days} = 2{,}800\left(\frac{W_c}{W_w} - 0{\cdot}5\right).$$

Let

$$\frac{W_c}{W_w} = r;$$

the fundamental equation of absolute volumes for 1 cubic yard of concrete is:

$$\frac{W_a}{S_a} + \frac{W_s}{S_s} + \frac{W_c}{S_c} + W_w = 1{,}684{\cdot}8 \text{ lb.} \quad \ldots\ldots \quad (1)$$

but

$$W_c = \frac{W_a + W_s}{R} \quad \ldots\ldots \quad (2)$$

and

$$W_w = W_a\,P_a + W_s\,P_s + \left(\frac{W_a + W_s}{R}\right) P_c. \quad \ldots \quad (3)$$

therefore

$$r = \left(\frac{W_a + W_s}{R}\right) \Big/ \left(W_aP_a + W_s\,P_s + \frac{W_a + W_s)}{R}\,P_c\right) \quad \ldots \quad (4)$$

or

$$\frac{W_a + W_s}{R} = r\,\frac{W_aP_a + W_sP_s)}{1 - rP_c} \quad \ldots\ldots \quad (5)$$

Equation (1) can be written

$$\frac{W_a}{S_a} + \frac{W_s}{S_s} + \frac{W_a + W_s}{RS_c} + \frac{W_a + W_s}{rR} = 1{,}684{\cdot}8 \quad \ldots\ldots \quad (6)$$

Substituting equation (5) in (6),

$$\frac{W_a}{S_a} + \frac{W_s}{S_s} + (W_aP_a + W_sP_s)\,\frac{(S_c + r)}{(1 - rP_c)S_c} = 1{,}684{\cdot}8 \quad \ldots\ldots \quad (7)$$

Equation (7) now has only two unknowns, W_a and W_s, but if the degree of workability is known, then W_s can be found; after which

$$W_c = r\frac{(W_aP_a + W_sP_s)}{(1 - rP_c)}$$

and

$$W_w = \frac{W_c}{r}.$$

The weights W_a, W_s, W_c, and W_w when converted to absolute volumes will equal 1 cubic yard of plastic concrete and the sum of the weights equals the wet density.

The dry density is obtained by deducting P_a and P_s from the total weight.

The percentage voids is obtained by converting P_a and P_s to absolute volume. This will give a direct result since a unit volume has been used.

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Mr F. N. Sparkes said that he found it difficult to offer any constructive criticism, because, as the Author had said in presenting the Paper, it had been necessary to make statements in it without qualifying them in any way. Of the many debatable statements made, Mr Sparkes would confine his remarks to three.

The Author had said that there was only one water/cement ratio for a concrete mix, and had estimated that requirement by determining first the amount of moisture required to obtain maximum compaction in the sand, as sand; he had then assumed that for the coarse aggregate the thickness of the film of water round each particle was 0·003 inch, and finally he had added to those two the water required to hydrate the cement, which he had taken as 24 per cent of the weight of cement. Mr Sparkes's own feeling about those three methods of determining the water was, first, that the moisture required for full compaction of the sand, as sand, was not the same as that required by the sand when it was used as an ingredient in a concrete mix, since the particle distribution was bound to differ in the two cases. Thus the figure of 9 per cent which had been quoted in the Paper as valid for sand when used in concrete was not necessarily an acceptable one.

Secondly, from whence came the figure of 0·003 inch for the thickness of the water-film round the coarse aggregate? It seemed to Mr Sparkes that small differences in that figure might make considerable differences to the calculated minimum moisture requirement.

Thirdly, so far as the water required for hydrating the cement was concerned, he was confused, because that water was the total water required to hydrate the cement over a very long period of time and only a small amount was taken up by the cement during the plastic period when being compacted; the remainder formed water voids and served as a reservoir for the continued hydration of the cement after a period of years. Therefore he did not agree that all that water could be used in calculating the water necessary just to provide absolute compaction; the reservoir of water-voids prevented that. Would the Author give more information on those three aspects of determining the moisture?

The Author had proposed a new definition of the workability of concrete and measured it as the amount of coarse aggregate in a unit volume of concrete. It was not logical to measure work in units of lb. per cubic yard. Further, that definition took no account at all either of the effect of the shape of the coarse aggregate or the shape and grading of the fine aggregate on the workability.

He had done a few laboratory tests since reading the Paper, using a compacting factor apparatus for measuring workability. Taking the 6·2 : 1

mix, with 36 per cent sand and 0·48 water/cement ratio, and taking two mixtures where everything was equal except the shape of coarse aggregate, the mix with rounded coarse aggregate had given a compacting figure of 0·85 and the other, with the same quantity of angular aggregate, a figure of 0·76. That was an enormous difference in workability, and the Author's definition could not be valid since it took no account of the effect of the shape of the aggregate.

It had also been stated in the Paper that the matrix of sand, cement and water had no real effect on the workability, but Mr Sparkes, in common with other concrete technologists, considered that the shape and grading of the sand had quite an important effect on workability; that had clearly been shown by tests reported in the literature. He had done tests on two similar mixes using the same coarse aggregate, and had found that with river sand the compacting figure was 0·76, and for crushed sand for the same grading it was 0·70.

The Author's contentions with regard to volumetric and weigh batching were diametrically opposed to all modern thought on the subject. It did not follow that modern thought was necessarily right in all particulars, but Mr Sparkes would like more information on the question of the effect of batching on the variation of the concrete cubes. The only evidence in the Paper had been that the coefficient of variation of cubes taken from a continuous mixer (which was in effect volume batching), had been 23 per cent, whilst for weigh batching a coefficient of variation of 11 to 12 per cent had been quoted. That showed an enormous difference in favour of weigh batching, as indeed did many other published data, and it was up to the Author to substantiate his statement that "volumetric batching is more consistent both in strength and yield than weigh batching and requires less direct supervision."

Mr D. A. Stewart referred to the statement in the Paper that water did not affect workability, and showed a slide giving a curve relating the time of consolidation to water/cement ratio. The work had been done on a vibrating table, where it had been possible to measure the point at which full compaction was reached. A constant mix had been used; the weights of sand, cement, and stone had been kept constant and the water content and the water/cement ratio varied. At 0·35 water/cement ratio the mix had taken more than 12 minutes to compact; at 0·40 it had taken less than 3 minutes, while at 0·525 it was very nearly completely slumped and had gone off in 6 to 8 seconds. That had been a 6 : 1 mix; the maximum size of aggregate had been $\frac{3}{4}$-inch and it had been graded slightly. Because the time of consolidation obviously had some relationship to the work that could be done, since the vibration had been more or less constant, the water appeared to have had great effect, especially on going from 0·35 to 0·40, dividing the time by approximately 4.

Mr Stewart referred to a job where there had been reservoir work to be carried out. In the first instance it had been suggested that a 7 : 1

mix should be used to get a really dense concrete. As the coarse material 1½-inch to ½-inch aggregate had been used, and an ordinary concreting sand, what was usually termed a good concreting sand, with a lot of coarse particles in it between $\frac{3}{16}$-inch and 14 mesh. The first mix had been made out of 1 of cement to 2 of sand to 5·2 of coarse material. The complaint had been that there were points at which honeycombing occurred, and the contractors had asked whether they could put in more sand and had been told to do so, and so on the job that mix had been 1 : 2·2 : 5·2. It had just opened up and was completely honeycombed, and so more sand had been added, increasing the proportion to 2·3 : 5·2. One bit of good concrete had resulted, and so 2·4 of sand to 5·2 of coarse material had been used. That had looked better still, but it had not been quite right and the workability was going down rapidly. Mr Stewart had been asked his opinion, and had expressed the view that there was too much sand being used. A change had therefore been made to 1·9 : 5·3. The form had been stripped immediately after vibrating and the hose used to disclose the aggregates. It had not been a bad job, and the wet density was 152 lb. per cubic foot.

Obviously there had been more in changing the coarse-to-fine aggregate ratios than met the eye, and perhaps some of the Author's differences with him were in the amount of sand which the Author liked to have in a mix—between 30 and 40 per cent—whereas Mr Stewart did his best to keep it down to between 25 and 27 per cent, and by that means a very workable concrete was obtained with very lean mixes. He referred to some recent road construction, where an 8½ : 1 mix was at present being used, the specified strength being 2,500 lb. per square inch at 28 days. He had designed the mix, and had just received the results of the tests. He was ashamed to say that, although he had thought that his mix was right, the strength was 3,000 lb. at 7 days when he had expected only 2,000. He had now to use 9 : 1.

Dr A. R. Collins asked how the method of designing mixes which had been described in the Paper differed from that normally employed. The Author had referred to optimum moisture content based on a density curve, but, as far as Dr Collins could see, that was only the moisture content at which the Author could use the lowest water/cement ratio and still obtain full compaction. If he made the mix any drier it did not compact properly; if he made it wetter, the water made the mix less dense.

Some of the Author's success in obtaining the results which he had achieved seemed to be due to the fact that he had worked with a very limited range of mixes. The mix proportions did not appear to vary more than between approximately 1 : 6 and 1 : 8. It would be interesting to see his methods applied to a much wider range of mixes and with aggregates other than those used at Heathrow.

Would the Author clarify his remarks about the control of quality? If he really thought that weigh batching and good supervision were unnecessary Dr Collins disagreed most heartily with him, and suggested that he was doing a disservice to those who were attempting to improve the control of concrete in the field, including the Institution itself which had a Committee considering the subject.

Was it now the Air Ministry's view that weigh batching and strict control of concrete were unnecessary ? That question was particularly important since the Author came from Heathrow, where much pioneer work on the control of quality on a large scale had been undertaken, as reported to the Institution in 1946.[1] Did the Air Ministry now believe that the time and money spent in developing the methods of controlling the concrete at Heathrow had been wasted ?

Mr E. E. Morgan referred to the Author's statements, on p. 18, that a concrete mix could not be fully compacted except when the water content was at its optimum value, as for soils ; that the strength of concrete appeared to depend on that optimum water content ; and that Abram's formula applied only under those conditions with full compaction. The optimum moisture content for soils, Mr Morgan said, depended upon the apparatus used to determine that value. For instance, if a certain value were obtained when using the Proctor apparatus, a lower value would be obtained when using the modified Proctor, having a heavier hammer and a longer drop. In other words, the value of the optimum water content for soils depended upon the amount of applied work.

Concrete mixes did have optimum values, but not in the sense implied in the Paper. For example, if workability were plotted against fine aggregate content for a 1 : 6 mix (by weight), consisting of 1½-inch river aggregate, tests had shown that for a water/cement ratio of 0·45 the workability increased as the fine aggregate was increased from 25 per cent to about 28 per cent and then it decreased when that percentage was increased. For a water/cement ratio of 0·50 the maximum workability occurred at about 32 per cent and for 0·45 it occurred at about 36 per cent. That did not mean that the mix could not be fully compacted if the fine aggregate were not 36 per cent, but that more work was required for compaction. Although the figures given depended upon the type of aggregate used, they showed that there was an optimum value for each water/cement ratio, but only in the sense that at that value the least amount of work was required to produce full compaction.

When describing his method of design the Author had said that the strength of the mix depended upon the percentage of voids, and had calculated those voids as being equal to the absolute volume of free water. That free water the Author had defined as all in excess of approximately 24 per cent of the weight of cement, that percentage being the quantity of water which was required to enter into chemical composition with the cement to obtain complete hydration. To obtain the quantity of water required to hydrate the cement, and that of the free water, the Author had dried the concrete artificially and had compared the wet and dry weights. Mr Morgan questioned the accuracy of that method for obtaining the air void percentage, and in that connexion he referred to a paper by Mr T. C. Powers [1] on cement hydration in relation to the curing of concrete. Powers had stated that for every pound of water required to hydrate the cement an equal quantity of water was locked in the cement gel. His experiments had indicated that 22 per cent was approximately the amount of water required to hydrate the cement (the difference between 0·22 and the Author's 0·24 was immaterial) and they had shown that a water/cement ratio of 0·44 was the least that could be used to make possible—not necessarily to achieve—full hydration of the cement. Powers had described that water as being non-evaporable. If enough heat were applied it would be possible to dry off that water, as the Author had done, but under ordinary conditions of use, in runways for instance, concrete was not subjected to artificial heat and much of the water which the Author had considered to be free water was not really free to evaporate. The absolute volume of the water locked in the cement gel should therefore be added to that in combination with the cement. With that method of calculation the reduction of the air voids was considerable. For example, in a 1 : 6 : 0·50 mix the air voids were less than 2 per cent instead of approximately 8 per cent as calculated by the Author's method. Regarding that question of voids and strength, it should be remembered that concrete cubes were in a moist condition when tested and that it was unlikely that runway concrete ever became completely dry.

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[1] George Graham and F. R. Martin, " Heathrow—The Construction of High-grade Quality Concrete Paving for Modern Transport Aircraft." J. Instn Civ. Engrs, vol. 26, p. 117 (April 1946).

In the Appendix the Author had given the various steps for the design of a concrete mix. Mr Morgan had been through that carefully and, although he did not agree with that method of design, he tried to see how, if he were coming fresh to the work and that was given to him as the usual method, he would obtain the appropriate mix proportions to suit a given set of conditions. The Author had said, quite correctly, that the two essentials for design were the strength and the workability. His formula for strength was the same as that given by Professor Walsh [2]

$$S = A\left(\frac{c}{W} - B\right)$$

where S was the strength required, $\frac{c}{W}$ the cement/water ratio and A and B were constants, having values which depended upon the quality of the concrete, its age, and the curing conditions. Walsh had included typical values for A and B, both for ordinary and rapid-hardening cement. The Author had given 2,800 and 0·5 as values for A and B with, on p. 26, only a very brief comment as to their suitability. No values had been given for rapid-hardening cement. Being an empirical formula, some explanation regarding its source and also its useful range should have been included in the Paper. In the Appendix the Author had used the symbol "r" for cement/water ratio, but on p. 31 he had used the same symbol for water/cement ratio. The point was a small one but it might cause confusion.

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[1] T. C. Powers, "A Discussion of Cement Hydration in Relation to the Curing of Concrete." Proc. Highway Res. Bd., Washington D.C., vol. 27 (1947).

[2] H. N. Walsh, "How to Make Good Concrete." Concrete Publications Ltd., London, 1939.

The second requirement was the degree of workability to be inserted for W_a in formula (7) on p. 37. Although the Author had measured workability by the quantity of coarse aggregate per unit volume, he had given no yardstick to indicate what would be suitable values for, say, medium or low workability, both degrees being referred to in the Paper. Information on that point was essential before the mix could be designed. Mr Morgan did not ask for a suitable value for high workability because the Author had expressly stated on p. 18 of the Paper that a different formula was required for concrete having an excess of free water, resulting in a slump, but he had not included in the Paper any formula for that class of material. Without it the position of the designer, using the Author's method, would be difficult because practical considerations often made it necessary to use the wetter mix. The Author had obviously carried out much experimental work when developing his method. Could he make any suggestions?

Mr A. J. Hill welcomed the point made in the Paper about the desirability of simplicity in methods intended for use in the field. Methods which were too complicated would not lead to good production and output. The methods must be understood by all the people in the field who had to apply them, and not by only one or two. He also welcomed the statement that it was not necessary to have an army of experts to get good results. He noted that the Author still relied on visual inspection when he got his result, as a check on the method.

There were many works where the specification gave the mix of the concrete in complete detail, and it gave the strength which was to be the outcome of that mix and left no variable quality or quantity to deal with the circumstances which arose on the site, with the frequent result that either the contractors did not get the concrete which they had bargained for or the engineer did not get the result for which he had hoped. Was it the Author's opinion that, provided he could get the concrete in the way he had indicated, it would be better if contractors were given the main specification of the strength of the concrete, and it was their responsibility to deal with matters of design and proportion of mixes? If so, what controls would the engineer require in order to satisfy himself that he was getting the concrete for which he had asked and for which he was paying?

Volume batching appeared to be, in the Author's opinion, a means of getting better control of the batching of concrete. His argument was apparent: that if there was variation in water content, with weigh batching it was necessary to alter the water and each of the aggregates each time, whereas with volume batching there was only the water quantity to alter. So far as Mr Hill knew, there were at present no mechanized concrete mixing plants which would give volume batching with that method of control, and no doubt there was scope for the review of the design and lay-out of concrete mixing plants, to give the Author the practical application in the field which his experience had indicated to be desirable.

The Author had referred to "no slump" concrete, and had said that, in the case of pumped concrete, sufficient water must be added to the minimum water to provide lubrication for the delivery pipes. That, he had said, could be calculated by adding a quantity of water represented by a film 0·003 inch thick on the lining of the pipes for each unit volume pumped. Did that mean that extra water had to be added to "no slump" concrete to take care of other matters which the concrete experienced between the mixer and the position in which it was placed? It would be a help to contractors to know that a slight moistening to deal with the absorption of reinforcement bars was recognized, so that they would not be frightened by the absolute zero slump. In other words, was "no slump," in the Author's opinion, the state of the concrete when it reached its final position? Lastly, if the Author obtained the concrete which he said that he did—and Mr Hill had personal experience of certain cases mentioned—would he favour removal from the specification of the ban on the dropping of concrete from heights of more than, say, 5 or 6 feet?

Mr N. F. Truscott gave the assurance that the Author's statement that the opinions expressed in the Paper were his own, and did not necessarily carry the endorsement of the Air Ministry, was perfectly true. The Air Ministry were still in favour of weigh batching for the accurate control of concrete, and that should be made quite clear.

He felt that in preparing the Paper the Author had tended to argue from the particular to the general. At London Airport he had a site with probably the nearest approach to ideal conditions for concrete making which could be found in Great Britain. The natural aggregates on the site were excellent. Mr Truscott felt that the Paper would have been very different, and probably would not have been written at all, if London Airport had been in Lincolnshire or in North Norfolk, where the aggregates were far less suitable.

In discussing the question of moisture content of the aggregates, in one part of the Paper where it had been intended to show that volume batching was sound practice, it had been stated that the moisture content of a large stock pile became stabilized within 1 per cent after 24 hours; but later, when giving the difficulties of weigh batching procedure, the Author had said that the problem of moisture control was one of the major factors against weigh batching. He could not have it both ways. In

Mr Truscott's limited experience, it was not unusual to get variations of up to 2 per cent in moisture content in a very large stock pile, depending entirely on where the dragline operator happened to pitch his bucket; and it could be shown quite simply that a variation of 2 per cent in the moisture content of the fine aggregate in, for example, a 4 : 2 : 1 mix, could lead to an increase in the water/cement ratio from say 0·48 to 0·53. Taking Abram's curve—he still believed in it, despite what had been said in the Paper—that would mean a loss of strength of approximately 9 or 10 per cent, which was a very serious matter. According to the Author that could be disregarded, because the variation in fine aggregate moisture content was negligible.

The Author had made one or two rather startling statements. He had suggested that the mixer driver was able to control the water/cement ratio to the second place of decimals by looking at the mix. In Mr Truscott's opinion it was difficult to control it to the first place of decimals by appearance only. An even more remarkable contention was that if the mix supplied to the vibrator was too workable, or the contrary, word was at once sent back to the mixer that the water content should be adjusted.

It had been clearly shown in *Fig. 6* that a stronger concrete would be obtained with a leaner mix. In Mr Truscott's opinion it was correct to say that with full compaction it was possible to get an equally strong concrete with a leaner mix, but he thought that a much more important way of interpreting *Fig. 6* was to read it horizontally, rather than vertically, and to show that a reduction in water/cement ratio at a constant aggregate/cement ratio would give a more rapid decrease in percentage voids, and therefore an increase in strength, instead of interpreting it in the way that the Author had chosen. If the aggregate/cement ratio were kept constant and the water/cement ratio reduced, a far more significant increase in strength would be obtained than by merely increasing the aggregate/cement ratio and making a leaner mix of it.

Mr. Truscott thought that Table 1, which was a record of a number of trial mixes purporting to prove those facts, had rather disproved them if carefully analysed. Taking the first two items in that Table, a slight increase was shown in the 7-day strength of the 8 : 1 mix when compared with the 6 : 1 mix, but he contended that that was no more than would be expected with the normal factors of variation: it happened in that case to come out at 200 lb. per square inch higher. Only those figures in the fifth column could be used, those at natural compaction; it was not possible to convert to 100-per-cent compaction. The principal reason for the weakness of a lean mix was that it could not be properly compacted.

The last two items in the same Table had purported to show that with the voids kept constant the change in strength was negligible; but in actual fact the change in natural compacted strength, as distinct from 100-per-cent strength, was very much greater than in the first two examples and in favour of the richer mix. Those two bore out what he had suggested, that the aggregate/cement ratio should be kept more or less constant. The Author had in fact increased it, but a reduction in the water/cement ratio resulted in a stronger concrete, and much stronger than any gain which might have been obtained by merely weakening the mix with a constant water/cement ratio.

Mr G. C. Wilson observed that doubts appeared to have existed as to the most efficient method of compacting the gravel at London Airport, as was shown by the account on pp. 7 and 8 of the four procedures which were employed, and the Author's disagreement with the suggestion that smooth-wheel rollers gave the best results with that type of material. The Author referred there, presumably, to the results obtained from the careful tests carried out by the Road Research Laboratory. It was a pity that no details had been given of either the complete grading of the gravel or of the rollers employed on the Airport, to indicate whether or not a direct comparison could be made between the compaction results achieved there and those reported by the Road Research Laboratory. Mr Wilson thought it was possible that the suitability of any particular type of roller depended on those physical characteristics; in fact it might be that when a medium-weight machine was used a certain type might be more effective than a second, but that when heavier models of the same machines were employed under the same conditions the second type might prove to be the better.

He found some support for that idea in a comparison between the Road Research tests on their gravel—sand—clay on which standard light- to medium-weight sheepsfoot rollers had given relatively poor performances, and some tests carried out in the U.S.A. with ultra-heavy rollers of the same type on a sandy gravel which had indicated that very satisfactory densities had been obtained. It might be, however, that a strict comparison was not valid when all the factors were taken into account.

One of the drawbacks of using sheepsfoot rollers, particularly on gravel and sand, was that a layer of loose material of appreciable thickness, depending on the length of shank of the feet, was left at the top, irrespective of the compaction obtained at depth. Whilst that might not be a grave disadvantage in the lower strata of a deep fill, it made control testing difficult and necessitated the use of another roller for finishing. For this purpose there was no doubt that the heavy rubber-tired rollers gave excellent results, though they might be criticized on the score of producing a ridged appearance on the surface. Under favourable conditions, properly controlled heavy pneumatic-tired earth-moving equipment could be most effective in compacting the bulk of the fill, but was not recommended for finishing operations.

The Author was somewhat inconsistent in his comments on the optimum moisture content. He went to the trouble of devising a laboratory method for the rapid determination of one value of that property, but did not employ any measures intended to correct the moisture content to the fill or to relate the optimum value to the plant used, although on p. 16 he had stated that rolling must be carried out at the optimum moisture content necessary for the type of roller used. It was possible, however, that with the gravel at London Airport the optimum moisture contents for rollers were all similar, and the same as the moisture content of the filling as dug.

It had been stated on p. 7 that the specification required the gravel to be laid in layers not exceeding 9 inches in thickness when compacted, and later, presumably as the result of site experience, only 8-inch layers were specified. Was that modification really significant, and on what data was such a critical adjustment based ? Later in the Paper it had been stated that a minimum figure for compacted density of 128 lb. per cubic foot was given. Had that figure been included in the contract documents, or did it represent the degree of compaction which was to be to the satisfaction of the engineer ? How had that figure been selected ? Would the Author describe any methods by which the compaction requirements had been assessed ? Had any attempt been made to relate the stability or any other property of the compacted fill with density, or did that target density merely reflect the degree of compaction which should have been obtained when the plant provided by the contractor was used most efficiently ?

Had anything been included in the specification relating to the desirable characteristics of the compaction plant, or had any recommendations been made as to the technique to be adopted ? Also, had the specifications for later contracts differed from the earlier ones ? Finally, could the Author explain how the items covering the compaction of gravel filling had been billed ? Had any separate allowance been made for the addition of water, and—the most important factor from the contractor's point of view—had the contractor been satisfied with the system adopted ? He felt that the the use of an indeterminate amount of " thorough ramming " in the compaction test was a retrograde step. Even in the Proctor compaction test the amount of work done was a known quantity. Further, in the estimation of the optimum moisture content for the entire material, how was that calculated when two of the three constituent fractions had more or less straight-line moisture/density curves exhibiting no peaks of density to which the individual optimum moisture contents could be related ?

It was Mr Wilson's experience that in the field the accuracy of the density measurements depended not so much on the accuracy of weighings but on the care with which the trial hole was excavated and then filled. A further source of error was the use of too small a hole. On a contract with which his company was associated a hole 12 inches in diameter by 8 inches deep was used in material consisting largely of particles up to 3 inches in size. The loose top gravel was first removed to a depth of several inches so that the test was made in filling which was representative of the fully compacted material. Fine gravel, graded between $\frac{3}{16}$-inch and

$\frac{1}{16}$-inch, was used in conjunction with a large pouring cone to measure the volume of the hole. Moisture contents were determined directly by oven drying.

Referring to the statement on p. 8 that "It is not a practical or economical proposition to try to accomplish laboratory results in the field" Mr Wilson registered complete disagreement with it as a general expression of principle. Surely it was only by the reproduction in the field of the improvements developed in the laboratory, through the evolution of new techniques and equipment, that the quality of full-scale works would continue to be raised.

The following extract from an article describing the construction of a new highway into Chicago refuted the Author's claim: "We have closely approached the production of 'laboratory' quality concrete in the field, without the job requirements making the cost prohibitive."

Mr W. A. Lewis said that although he agreed that at Heathrow the pneumatic-tired roller was probably the most satisfactory type of plant to use in view of the general site conditions, where there were rather uneven ground conditions and perhaps a large number of tractors available to tow pneumatic-tired rollers, he did not agree that that would necessarily apply to all large contracts, as suggested by the Author. Evidence which had been obtained at the Road Research Laboratory had shown that the heavy smooth-wheel roller was able to produce as good compaction on gravel as other types of roller in common use, and its lower output as compared with the pneumatic-tired roller was perhaps more than compensated for by its much lower cost of operation. With regard to the use of sheepsfoot rollers, the information which had been obtained from research was that that type of roller was quite unsuitable for the compaction of gravel of the type occurring at Heathrow.

The Author had referred to the thickness of gravel which would be compacted with different types of roller. On what information had he based his opinion, since in Mr Lewis's experience it was an extremely complex matter to obtain precise information on that point? There was usually a gradient of density through a compacted layer of soil no matter how it was compacted, and it was therefore only possible to speak of average densities in a given depth of the soil. In investigations by the Road Research Laboratory on the compaction of a sandy gravel of similar grading and moisture content to that occurring at Heathrow, no difficulty had been found in producing densities of 134 lb. per cubic foot when using an 8-ton smooth-wheel roller, and loose layers 9 inches thick compacted to 6 inches.

On p. 2 the Author had remarked that it was frequently stated that the optimum moisture content of gravels was approximately 7 per cent. Since the Author had, perhaps, been referring to information given in publications of the Road Research Laboratory, it should be mentioned that the soils which the Laboratory had tested with optimum moisture contents of 7 per cent were gravel-sand-clays or hoggins, and were not the relatively clean gravel-sand of Heathrow, which would obviously be expected to have a much lower optimum moisture content than a hoggin.

Mr J. M. Fisher remarked that the Author was apparently of the opinion that volume batching, as carried out on one job with which he was connected, gave results comparable with any attained by weigh batching. On that particular job the results attained—an average strength of 6,956 lb. per square inch and a coefficient of variation of 8·72 per cent—gave a standard deviation of 605 lb. per square inch at 28 days. Although Mr Fisher had no up-to-date figures for work at London Airport, he believed that the average strength for the high-grade paving concrete was being maintained at about 5,400 lb. per square inch, and the coefficient of variation at about 9·50 per cent. That gave a standard deviation of approximately 510 lb. per square inch, which he submitted indicated a much closer degree of control. The volume-batching plant used on that job was a very elaborate one, far more elaborate than was often used on constructional work, and could be considered a very good second best to a weigh-batch plant The list of jobs where the degree of control over the quality of concrete was equal to or better than that at Heathrow was increasing, but in every case the results had been obtained with weigh-batching equipment.

With regard to the all-important question of the yield of concrete from the two classes of plant, the Author's argument would be very much strengthened if he were to explain why the yield from weigh-batching plant could not be accurately assessed within 5 per cent. A series of tests recently carried out to check the actual weights of materials dry-batched into trucks had indicated an average discrepancy within 0·50 per cent of the true batching weight, and a maximum discrepancy of approximately 3 per cent. The yield of concrete containing a fixed amount of water per batch would therefore be to all intents and purposes constant.

The Author had referred to the adoption of a properly designed mix of 1 : 7½ by weight, with a water/cement ratio of 0·55, produced in a continuous mixer and giving results comparable with those obtained from a weigh-batched mix of 1 : 6·2, with a water/cement ratio of 0·48, where there was a very high degree of control. If by "comparable" he had meant that the results were similar, that was, to say the least, an extremely important finding, especially when the financial implications were considered. The difference in cost of the materials per cubic yard of the two concretes was probably about 3*s*., and the potential savings on even a small job might be considerable.

On the subject of continuous mixers, Mr Fisher would refer the Author to a Paper [1] presented to the Works Engineering Division of the Institution by Messrs Lumbard and Wright during the 1949–50 Session. They had quoted the results of a long series of tests on concrete production in continuous mixers under well-controlled conditions, and compared them with the results obtained from leaner mixes using inferior aggregates produced in a weigh-batching plant. The variation in the strengths of the concretes from the continuous mixers was more than twice that from the weigh-batch mix, and the average strengths were very similar. Although costs had not been disclosed, that of the material in the weigh-batched concrete must have been very considerably lower.

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[1] Donald Lumbard and A. A. Wright, "New Steelworks, Port Talbot—Construction of Works." Instn Civ. Engrs, Wks Constr. Paper No. 17, 1950.

Mr. Fisher noted that the Air Ministry prohibited the use of continuous mixers for the production of high-grade concrete. That prohibition had probably arisen as a result of unfortunate experiences during the war, despite attempts which had been made to improve the quality of the concrete delivered from such mixers. Had everybody who had been using such mixers been using them wrongly, or had everybody been using the wrong mixes ? Could the Author give information on that point ?

The curve shown in *Fig. 8* of the Paper was very similar in form to the well-known curve relating the strength and density of concrete which appeared in Road Research Technical Paper No. 5. To prove the reasonableness of that curve, the Author had analysed the results of a series of strength and density tests on a number of cubes of concrete having mix proportions of 1 : 6·2 and a water/cement ratio of 0·48, and had obtained the curve shown in *Fig. 9*. On the basis of the specific gravities quoted for the various constituents of this concrete, its maximum density when no air voids whatever were present was 149·2 lb. per cubic foot. Tests on concrete having similar rheological properties to that mix, carried out by means of an air meter, had indicated that it was virtually impossible to compact such a concrete by means ordinarily available to such an extent that air voids were completely excluded, so that even a density of 149·2 lb. per cubic foot could not be attained in practice ; yet a third of the results quoted had densities greater than that figure. It might well be that the densities per cubic foot had been calculated as being eight times the weight of a 6-inch cube, whereas if the cube as cast had a side of 6·04 inches, as might easily be the case, the bulk density should be calculated as 7·82 times the weight of the cube. Could the Author explain more fully the means by which the curve had been obtained ?

The Author, in reply, said he had thought that the Paper might evoke a lot of discussion, and in fact a lot of useful information had been given, but he thought that the Paper had been misread in many ways. He had not said anywhere in the Paper that volumetric batching was better than weigh batching ; he had tried to show that in any contract there was a place for volumetric-batching plant. A weigh batcher was not always available and there was no need to hold up work if a volumetric batcher was on site.

Mr Sparkes had asked about the origin of the 0·003-inch thickness of water film. The Author had shown, in *Fig. 5*, the moisture curves for gravel plotted adjacent to a curve for spheres having a film of water 0·003 inch thick, and which gave figures which could be used in practice.

The figure of 9 per cent moisture for sand was an arbitrary one and depended upon the fineness modulus of the sand. For $1\frac{1}{2}$-inch to $\frac{3}{16}$-inch stone, sand, and cement the moisture figures of 1 per cent for stone, 9 per cent for sand of medium fineness, and 24 per cent for the cement gave a no-slump mix. Furthermore, when the sand had a moisture content of about 17 per cent, segregation would take place. That was a very simple test to do, and a large number of such tests had been carried out at London Airport with predictable results in every case. It was agreed that the hydration of cement took place over a long period; the Author had used that term but had qualified it by calling it apparent hydration, to enable him to predetermine the voids in a mix, that being necessary to enable an assessment to be made of the compaction. Mr Sparkes would, of course, obtain a different compaction factor between rounded and irregular aggregate. The Author had indicated that at the bottom of p. 27. The difference between those two shapes would be about 200 to 300 lb. of stone in a cubic yard of concrete. The Author had not stated that the matrix of sand, cement, and water had no effect on workability but quite the contrary, as given on p. 17 of the Paper.

Mr Stewart had referred to honeycombing in a mix he had made, which was obviously a gapped grading with a very low percentage of sand. A low percentage of sand was required for a gapped grading, but not lower than 26 per cent, and with the water/cement ratio used excessive voids and honeycombing were to be expected.

The Author would have calculated the correct proportions of the aggregates as indicated in the Paper.

Dr Collins had stated that the Author might have done a disservice to weigh batching, but that was not so. Weigh batching was perfectly satisfactory, but it was more difficult to control and usually cost considerably more than volumetric batching. In weigh batching there were hidden costs, because both the engineer and the contractor had to provide a number of experienced men to ensure effective control. The Author was of the opinion that more consideration should be given to the economic point of view, having in mind the question of yield. If the contractor had to price his concrete on unknown factors then those factors had to be paid for. It was desirable to collect as many as possible of the known factors and that was what the Author had attempted to do in the Paper. Today, contractors tended to reduce their prices because they were beginning to know more about the yield of concrete. Some of them had got beyond the stage of adding up the volumetric ratios and applying a shrinkage factor. The Author was not attempting to give any pure theory but was giving the figures which he had found most suitable in practice.

Mr Morgan had agreed that concrete mixes did require optimum water, but had appeared to assume that the Author had suggested that a concrete mix of a fixed aggregate/cement ratio had only one optimum. The Paper was quite clear and *Fig. 10* showed that the optimum varied with the percentage of sand, which was the same as the statement made by Mr Morgan. With regard to workability, the method given in the Paper was quick and any designer could use it by inserting any figures he wished. The figures given in the Paper applied to straight line grading. A variation of about 200 lb. on each side would give approximately high and low workability. Slight variations would also have to be made for any change in the grading of the sand, but to a large extent that was taken care of in the formula.

For a change in shape the range was about 400 lb. between crushed rock and rounded pebbles, allowing for variations in specific gravity. The formula given for strength was of the same form as that given by Professor Walsh, but the slope of 2,800 could be applied to general good workmanship by the contractor, and had been found to apply to most contractors on the site. The figure of 0·5 applied to no-slump concrete with maximum compaction, and increased when more or less than optimum water was used.

Mr Hill had raised a point regarding the usual method of over-specifying a mix. The Author's opinion was that strength and yield of cement per cubic yard of concrete were all that was needed. That would compel contractors to use more efficient machines and tend to the better control of concrete. It might also lead to a better understanding of gapped gradings, thus promoting the use of a lower percentage of sand and a more durable concrete. At the present time, good quality concrete could be produced more cheaply by using coarse and fine aggregates as raised from the pits; the control of workability being in the correct proportioning of the two sizes of aggregate. Because, by that method, there was a fair proportion of intermediate sizes that led to a high percentage of sand to promote workability, and until better control of gapped gradings could be devised the cheapest good quality concrete could be produced by the methods given in the Paper. Mr Hill had also referred to dropping concrete from a height. The Author had never experienced any difficulty in doing that. Concrete having medium workability, with practically no slump and with a slightly increased proportion of intermediate fines, would not segregate when dropped from heights of up to 30 feet. At London Airport, reinforced concrete walls 20 feet high and 18 inches thick were being cast in one lift for a length of 25 feet.

The Author was of the opinion that the usual clause in specifications which prevented dropping concrete could be modified if the contractor could be persuaded to use a no-slump mix.

The Author had found that a no-slump mix could be used in reinforced concrete providing that the reinforcement was first wetted. If the reinforcement was not wetted beforehand then the maximum slump should not exceed ½ inch to ¾ inch. For pumping concrete, if medium workability was used, then about ½-inch to ¾-inch slump was all that was necessary, but a straight-line grading should be used with a tendency to a reduction in the intermediate fines.

Mr Truscott, in his remarks had shown that he had not read the Paper very intently. The Author had not stated that volumetric batching was better than weigh batching but that both methods had their place on a contract. The statement that London Airport was on a site with probably the nearest approach to ideal conditions for concrete making showed lack of experience, because, the Author suggested, a much better concrete could be obtained with granite or limestone, and he could assure Mr Truscott that just as good a concrete could be made with the rounded aggregates of Lincoln and Norfolk as with the irregular shapes found in the Thames valley: it was purely a matter of proportioning the coarse with the fine. The question of water in the sand was the main problem in trying to control a mix, and nowhere had the Author stated that a 2-per-cent variation in the sand was negligible and could be disregarded. On p. 4 the Author had referred to a 2-per cent variation in the total water, and a mixer driver could control the mix by eye up to about that limit and had, in fact, done so since the beginning of the contract, as was also reported by Graham and Martin in Airport Paper No. 1.[1] The necessity for the adjustment of a mix by eye was usually caused by variations in the fineness modulus of the sand, and should the mix be unsuitable at the placing end word was sent back to the mixing plant by the first available concrete lorry driver, and the mix was readjusted.

Mr Wilson and Mr Lewis had both raised some interesting points in connexion with types of rollers suitable for the compaction of gravel. The statements made in the Paper were based purely on experience gained on the site. The smooth-wheeled roller had not been used on account of it's lower output and because it usually produced unstable conditions. An answer had never been found to that problem and pneumatic-tired rollers were in general use for bulk filling. Smooth-wheeled rollers were used only for the final 2-inch topping to obtain true levels. There was no significance in the change of thickness of gravel layers between the two contracts; it was purely a matter of depth of fill, which in the second contract was a multiple of 8 inches against 9 inches in the first contract. The minimum density specified in the contract was 128 lb. per cubic foot and was based on the natural density of the undisturbed gravel which had been found to be 126 lb. per cubic foot. An additional 2 lb. was thought to be a reasonable minimum to specify and was in fact exceeded in the average, as stated in the Paper. Density was the only criterion owing to the difficulty of trying to relate it to bearing capacity. An attempt had been made to do that but no relationship could be discovered. In a series of tests a density might be 128 lb. per cubic foot with a K-value of 2,000, while a density of 135 lb. taken a few yards away would give a

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[1] George Graham and F. R. Martin, "Heathrow, The Construction of High-grade Quality Concrete Paving for Modern Transport Aircraft." J. Instn Civ. Engrs, vol. 26, p. 117 (April 1946).

K-value of 600. Sieve analyses gave no guide but high moisture contents frequently gave lower *K*-values.

Mr Fisher had referred to the 7·5 : 1 mix from the continuous mixer. When stating that that gave comparable results, the Author had meant that it gave him an average test cube stronger than he had been getting on the runway concrete, but that was not to say that continuous mixers could be used to turn out 1½ million cubic yards of concrete for runways without getting a much greater coefficient of variation. That mix related only to a road about 6 miles in length, 20 feet wide and 8 inches thick, and very satisfactory results had been obtained. The lowest cube-strength had been about 4,500 lb. per square inch and the highest 6,500, and for a road of that size about two hundred and eighty test cubes had been taken. That did not compare with the main runways, from which approximately twenty thousand cubes were made which had given a much better curve. The Author considered that standard deviations and coefficients of variation should be applied to the sampling of mixes rather than cubes. A far better control could be obtained by taking as many cubes as possible in 1 day's mixing and finding the standard deviation, which would then give an indication of the probable lowest cube-strength based on the total number of mixes required for the contract. Mr Fisher had also appeared to think that the Author was advocating volumetric batching in place of weigh batching. That was not the case, but the Author still contended that a much more consistent yield was obtained with volumetric batching owing to the inherent difficulty of assessing the contained water. Variations in grading, especially in the fines, tended to upset workability and in attempting to rectify that water only was added or varied. That, of course, was wrong where weigh batching was concerned because the weights of the aggregates should also be altered to maintain constant yield. In volumetric batching changes in grading of the fines were not so noticeable, because the effect was for the variables to cancel one another out. In the Author's experience over a large number of contracts he had always found that the yield from volumetric batching was closer than that of weigh batching, whilst a slightly smaller standard deviation was obtained with the latter. The Author was in agreement that continuous mixers were unreliable, but on the road mentioned the results obtained were good, the concrete was considerably cheaper than the runway concrete, and never gave a cube strength lower than 4,000 lb. per square inch specified for the runway concrete. The Author could only suggest to Mr Fisher that if a mix were designed to suit the method of placing a continuous mixer could be used, but better results would be obtained from a batching plant which had finer controls. With regard to his remarks on the densities obtained in *Fig. 9*, the Author agreed that the calculated density was 149·2 lb. per cubic foot. The method of finding those densities was by means of measurement with a micrometer. That may have given high results but the densities given could be reduced accordingly without altering the appearance of the curve.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

30 October, 1951

Lieutenant-Colonel R. H. EDWARDS, M.I.C.E., Chairman of the Division, in the Chair

The following Paper was submitted for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Maritime Paper No. 18

"The Design and Construction of the New Deep-Water Quays at Leith"

by

Matthew Cochrane White, B.Sc., A.M.I.C.E.

SYNOPSIS

The Port of Leith, although one of the oldest harbours in the United Kingdom, has been developed only during the last 150 years to accommodate large vessels by dredging and successive reclamations from a shallow foreshore.

The completion of the two new deep-water quays, which form the subject of the Paper, marks the first stage in the development of a large and important new area for future expansion. The quay walls themselves, built with reinforced-concrete caisson units, are of unusual design.

Reference is made to the character of the site which, together with the special purposes of the wall, influenced the arrangement of foundation caissons and the final design.

An account is given of the stability and ground-pressure investigations, and the assumptions used for all calculations, including the design of the reinforced-concrete caissons. Details of the temporary works required and the construction methods employed for building, launching, floating, and positioning the various units are fully described and illustrated.

INTRODUCTION

THE completion, in 1950, of the new deep-water quays at Leith has a special significance for the port, since it marks the first stage in the development of a new area and a long-term plan of expansion which, when completed, will double the capacity of the present system of docks. In addition, the berths are open to the tide, thus departing from the principle of enclosed docks with the many disadvantages of entrance locks, which are not only costly to operate and maintain, but are also restrictive and inconvenient for shipping.

The Port of Leith is situated on the south shore of the Firth of Forth, in latitude 56 degrees north, where a small stream known as the Water of Leith discharges into that estuary. The landward approaches are good, with excellent road and rail communications, which make it the natural collecting and distributing centre for the City of Edinburgh and the East of Scotland. Furthermore, the Firth of Forth, which opens on the North Sea opposite the countries of Scandinavia and North-West Europe, can be entered and navigated with safety by ships of any size, even when the most severe storms are experienced in the sea outside.

These geographical assets complete the natural advantages of Leith, and its development from a small fishing harbour to a seaport capable of accommodating ocean-going vessels has only been accomplished by a series of extensive civil engineering works within the past 150 years.

The Port of Leith is one of the oldest seaports in the United Kingdom and, until the beginning of the nineteenth century, the needs of shipping were met by quay walls or timber landing stages which extended for about ½ mile along both banks of the small tidal stream. The bed dried out at low water, and the course of the stream was continually shifting position in its progress over low sandbanks on the foreshore to the deep water of the Firth.

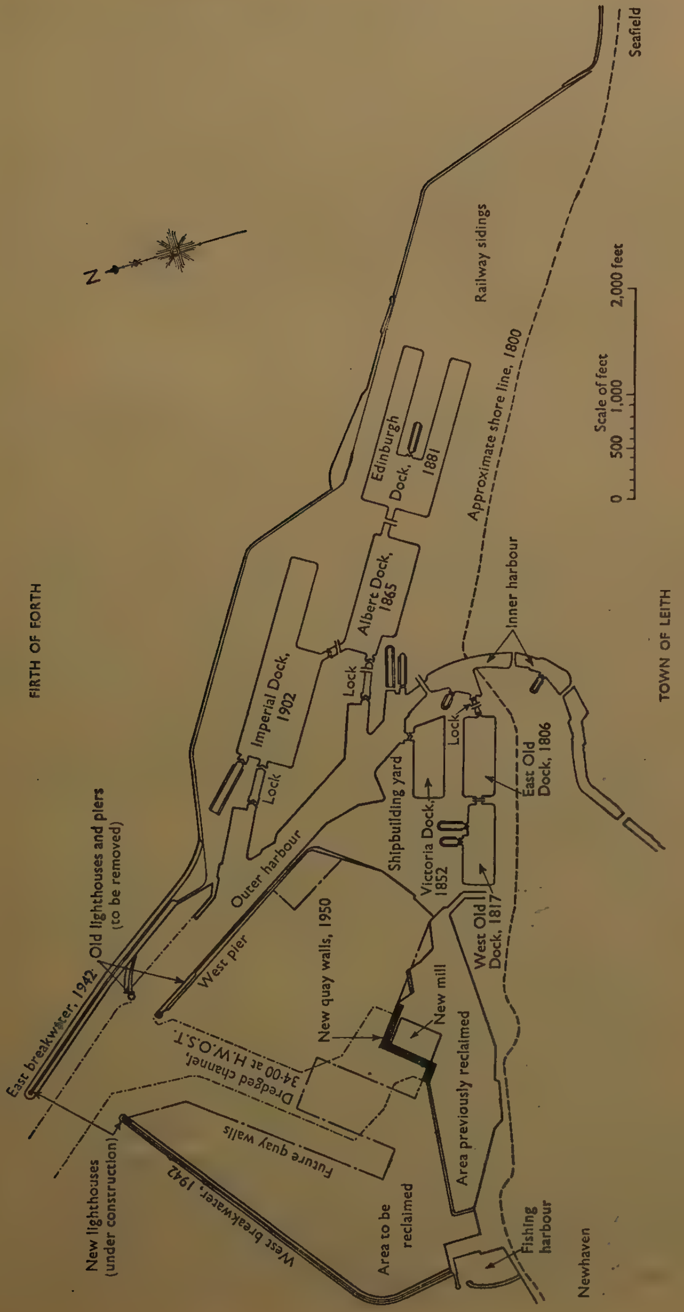
Expansion of trade with Europe and the Colonies led to the construction of the first enclosed docks, now known as the East and West Old Docks. These were completed in 1806 and 1817 respectively and were followed by extensions in the form of additional enclosed docks completed in 1852, 1865, 1881, and 1902, as shown in *Fig. 1.*

This century of progress witnessed the steady expansion in the trade of the country as a whole and coincided with the growth of industry, the development of the railways, and the transfer from sail to steam with the ever-increasing tonnage and draught of the steamship. Trade by 1913, particularly the export of coal and the import of grain, had expanded to such an extent that further extension of the quayage and improvement of the entrance had become a matter of urgency; the outbreak of the First World War, however, caused extensions to be indefinitely postponed. The subsequent depression precluded any capital expenditure on major works, and up to 1936 the improvements were confined to the equipment and modernization of facilities in the existing system of docks.

Owing to the proximity of the town to the shore line and the absence of a river with any considerable depth and volume of water, the Port of Leith had perforce been made to grow outwards into ever-deepening water by reclamation from the sea with expensive sea walls or breakwaters. Each new dock extension has, in the past, involved the construction of an additional and more expensive sea wall, which enveloped and rendered valueless the sea defences of the previous docks.

When the question of extension was reviewed in 1913, and again in 1934, that problem received very careful attention and resulted in new breakwaters being sited and built, on the Dutch principle, to enclose a large bay immediately to the west of the harbour entrance. These works, started in 1936 and completed in 1942, served the double purpose of providing a convenient and improved entrance to the whole dock system and

Fig. 1



THE HARBOUR AND DOCKS OF LEITH

enclosed a water area of about 250 acres, which was suitable for the successive development of quay walls as they might be required. Surplus dredgings, not required or unsuitable for the construction of the breakwaters, were pumped ashore behind a bank of boulder clay and formed a level area extending to about 32 acres at a height of 6 feet above the high-water level of ordinary spring tides.

This area of level ground, fronting a sheltered expanse of water, acquired great national importance in the following year, when it became the main centre for the construction of Mulberry Pierhead units. Thirteen pierheads and sixteen of the intermediate pontoons, representing more than 60 per cent of these vital components of the Mulberry Harbour, were completed at this site in time for the invasion of Normandy.

Inception of Scheme

After the Second World War a request was made to Leith Dock Commission by Messrs Joseph Rank, Ltd, for a site on which to construct a modern flour mill, with silos, warehouses, and other ancillary buildings adjacent to two deep-water quays. It was ascertained that good road and rail access would be required and that the buildings, which were to be of large dimensions, would extend to about 4 acres. One quay was to provide berthing facilities with modern intake plant for the import of bulk grain cargoes, whilst the other berth had to be suitable for the export of flour and other finished products into a smaller type of vessel employed on the coastal or European traffic. A quay apron with a width of 40 feet was required to accommodate the cargo-handling equipment and provide ample space for lanes of road vehicles. The quay walls, which were to be at right angles to each other, required minimum lengths of 500 feet and 340 feet with effective depths, below low water at ordinary spring tides, of 30 feet and 22 feet respectively.

These preliminary details showed the project to be of an importance and magnitude sufficient to warrant the construction of new deep-water quays to mark the first stage in the development of the Western Extension.

Site at Western Extension

An outline plan of future quay walls in the new area had previously been prepared and, on paper, it was a comparatively simple matter to select a site which could satisfy all the above requirements, but at that time there existed merely the level area of reclaimed ground fronting a sheltered but shallow stretch of water. The south shore-line was straight, formed by a bank of tipped boulder clay, and the depth of water varied from zero at the toe of the bank to the maximum of 18 feet at L.W.O.S.T. near the centre of the bay, where a channel had been dredged during the earlier construction of the breakwaters. Outwith the dredged channel the original sea bed was irregular, but generally lay only a few feet below

low water. A considerable asset was the success attained by the new breakwaters in limiting wave action inside the area, since, even in unfavourable circumstances, the maximum height of wave did not exceed 2 feet.

Considerable knowledge of the geological peculiarities and irregularities of the Western Extension area had already been gained from earlier works, but a more exact and accurate exploration was made by sinking a series of forty-two boreholes, to a selected pattern, over the area of the proposed quay walls and the adjacent berths. These confirmed that, underlying the top few feet of sandy silt, a bed of grey boulder clay of irregular depth would be encountered 9–14 feet below low-water level, with beds of rock, comprising shales and sandstones, at the level of the deeper berth. From the experience of dredging previously undertaken in the same area during the construction of the breakwaters, it was known that this clay was of an extremely tough nature, and contained a large number of boulders of exceptional size throughout the full depth.

DESIGN OF QUAY WALLS

Preliminary

Many factors may enter into the design of a quay wall, but the type and cost of the structure finally adopted will be influenced largely by the site conditions, the nature of the strata, and purposes for which the quay wall has to be built.

There are numerous types of quay walls or wharfs but, in general, it may be said that they fall into one or other of two main classes: piled construction or gravity construction.

From past experience it was known that the nature of the local boulder clay, at the relatively high level shown by the borings, would preclude any form of piled construction in the permanent structure. This automatically limited the choice to some form of gravity wall.

The first point of note about the site was that it lay under water of varying depth, except at one corner where it touched the reclaimed area. It was therefore decided that any excavation for the new quays would be carried out by dredger at the same time as the adjacent berths. In tough under-water materials a robust type of bucket-ladder dredger is usually the most economical plant unit to employ, and by this means, it was thought that the foundation depths could be reached, even in the top layers of shale or sandstone, without resort to a rock-breaker.

Consideration was given first to a design of wall which envisaged a normal arrangement for caisson-wall construction. The substructure would have been formed with a row of large hollow reinforced-concrete caissons built at special launchways, floated into position, and sunk end to end, on a prepared mattress of stone. Once in the final position, they would have been filled with either sand or concrete and the superstructure above low-water level completed with pre-cast concrete blocks or mass concrete, as might be preferred.

This design, although it has known disadvantages, would have proved quite suitable for an ordinary quay wall with the objects of maintaining a water frontage and facilities for marine traffic, but, it will be remembered, heavy buildings were to be constructed at a distance of 40 feet from the cope. The actual building loads from the front columns were not stated but, from the nature of the proposed constructions, it was known that they would be heavy. It was desired to incorporate foundation caissons of sufficient width to be capable of supporting the front portion of the building, which, apart from increasing the stability of the quay wall, would effect a considerable overall economy in the completed scheme.

This caused the abandonment of the first design, owing to the greatly increased cost of providing larger and wider continuous caissons with the practical difficulties associated with the construction and handling of such unwieldy structures. These preliminary deliberations, however, led to the adoption of a novel design, which comprised three types of floated caissons, conceived by the late Dr K. W. Mautner.

Extent and Description of Works

The site selected for the new quay walls occupied a central position in relation to future quays and, to avoid interference with traffic when later construction was required, it was decided that these new walls should be extended at each end. The final plans adopted involved the construction of two quays, with lengths fixed at 500 feet and 424 feet, forming the sides of a right-angled triangle, with the apex pointing due north into the water area and based approximately on the bank at the edge of the reclaimed land. An extra length of quay to the west required the construction of a further 120-foot length of wall with a re-entrant right angle, owing to change of direction.

Extensive dredging was required to provide an adequate approach channel, and depths of 30 feet and 22 feet below L.W.O.S.T. at the 500-foot and 424-foot berths respectively.

The triangular water area contained by the two new quays, extending to about 2 acres, had to be reclaimed, and temporary connexions to the reclaimed area were necessary at the ends of the walls to provide access from the land and to protect the banking from erosion.

The foundation caissons which project 2–3 feet above low-water level at spring tides, instead of being placed end to end, were turned transversely to the line of the quay and positioned at intervals in the manner of caisson piers for a bridge (see *Fig. 2*, facing p. 68). By this method it was possible to reduce the size and weight of the caisson shells whilst, at the same time, the quay wall itself could be economically widened to provide a 40-foot apron and a substantial foundation directly under the front wall of the buildings.

The reduced size of the ground caissons, all of which measured 45 feet by 15 feet in plan, made it economically practicable to incorporate a chamber for compressed-air working and so enable the final excavating and concreting operations to be carried out in the dry. With a compressed-air caisson it was possible after a visual inspection of the stratum to select, within limits, the best level for the foundation. Any soft spots encountered could be removed and, the more exact bearing thus obtained, permitted a high but safe unit bearing-pressure to be used for the design.

Centre-to-centre spacings of the ground caissons were 38 feet 3 inches at the 500-foot berth and 34 feet at the 424-foot berth, where the respective foundation strata were expected to be rock and boulder clay. The spaces between the ground caissons were filled in by a rock bank, with a front slope of 1 in 1½, and, at the critical levels above and below low-water level, by a reinforced-concrete back caisson supported on cantilever brackets and designed as a beam to withstand the thrust from the filling behind the walls. To prevent loss of filling through the rock bank the reverse face was covered by a screen of 2-inch broken whinstone placed to a minimum thickness of 3 feet.

Top caissons, comprising the superstructure to above high-water level, were of cellular box construction, and at the 500-foot berth, incorporated a subway for belt conveyors and beams to support the travelling grain intake towers. These caissons bridge between the ground caissons, leaving intermediate 5-foot gaps directly above a ground caisson and between each top caisson. These gaps were later made up with in-situ concrete, all carried out in the dry between tides. Figs 3, Plate 1, show sections and part elevations of the quay wall at the 500-foot berth.

Special advantages of the plan lay in the large amount of repetitive work, with a consequent economy of timber for shuttering and temporary works, and the possibility of economizing in man power by a high degree of mechanization.

The original plan envisaged the construction of eighty-one caissons to complete 1,044 linear feet of quay wall as follows :—

500 feet with berth dredged to 30 feet at L.W.O.S.T.
120 ,, ,, ,, ,, ,, 30 ,, ,, ,,
424 ,, ,, ,, ,, ,, 22 ,, ,, ,,

Owing to the delay in obtaining the special equipment for travelling, launching, and positioning the floated caissons, it was suggested by the contractor that an immediate start could be made to the permanent work by sinking ordinary land caissons from ground-level at the south-west corner where the quay walls impinged on the reclaimed land. This was agreed to, and three large reinforced-concrete caissons were designed to replace a total of fourteen floated caissons of various types, including a large double corner caisson. Because of the elimination of the caissons at the first corner, a further modification to the design was allowed for the exterior corner. In this case, a special steel shoe was built ashore and launched to act as a floating base for the large reinforced-concrete caisson to be built while afloat.

Particulars of the caissons are given in the Tables 1 and 2 in the Appendix.

Stability and Bearing Pressures

Before proceeding with the detailed design of the caissons it was necessary to investigate the stability of the proposed walls and to calculate the maximum pressures under the foundation caissons. Moments of the horizontal and vertical forces were calculated in the usual manner to determine the position and magnitude of the resultant, with the following assumptions for the horizontal forces acting behind the wall :—

(1) Full hydrostatic head between high- and low-water levels.
(2) Superimposed loading of 5 cwts per square foot on the backfilling.
(3) Full dry earth pressure down to high-water level.
(4) Immersed earth pressure below high-water level (voids : 45 per cent).
(5) Angle of repose : 30 degrees.

It was known that the foundations would be on shale or sandstone at a depth of 30 feet below low-water level, and on softer shale or boulder clay at the higher level. Permissible safe loadings on these materials were taken to be 8 tons per square foot on the rock foundations and 4 tons per square foot on the boulder clay. Samples of boulder clay, for soil mechanics tests, were taken from boreholes, but results obtained from the tests proved inconclusive owing to the difficulty of procuring truly representative undisturbed samples in such material. The tests did indicate, however, that the bearing capacity at 4 tons per square foot was very safe.

For the purposes of calculation, the spacings for the foundation caissons were taken at 38-foot-6-inch and 33-foot-9-inch centres for the deep and shallow berths, where the foundation materials were expected to be rock and boulder clay respectively. This left clear spaces of 23 feet 6 inches and 18 feet 3 inches between the caissons, and it is a feature of the design that, in these gaps below the level of the back caissons, the horizontal pressure from the filling is fully supported by the rock bank and not transmitted to the adjacent caissons.

In the case of the larger wall on a rock foundation, the calculations showed that the resultant would strike within the middle third and that maximum pressures would amount to 7·28 and 0·48 tons per square foot at the toe and heel respectively. These results were obtained with a loading which might be visualized when the quay walls and backfilling were completed with constructional work, in progress behind the walls, imposing an equivalent loading of 5 cwts per square foot on the reclaimed land.

This result was considered satisfactory, but further combinations of probable loadings, superimposed on the first case, were worked out to include :—

(1) Column loads of 200 tons from a building bearing directly above each foundation caisson, 3 feet from the landward edge.
(2) As in (1) plus load from a travelling grain intake tower on the quay wall.
(3) As in (2) plus loading on the quay wall from motor traffic between the building and grain intake tower.

The general effect of these additional loads was to improve the distribution of pressure, and in case (1) the respective toe and heel pressures became 6·8 and 1·2 tons per square foot. It will therefore be possible for the quay wall to support a much greater load from the building if this is required.

In the other case, with the caisson founded on boulder clay, the edge pressure at the toe was found to reach 4·62 tons per square foot, and therefore exceeded the permissible figure. This result was obtained by assuming the normal diagram for trapezoidal distribution of pressure which, in material such as boulder clay when near the limit of plasticity, may be modified to show a better distribution over the whole foundation and a resulting decrease of the maximum toe pressure. This calculation showed a reduction of from 4·62 to 4·28 tons per square foot at the toe, but a more satisfactory result was desired.

An obvious solution, only possible with a compressed-air caisson, was to increase the bearing area by undercutting the front portion of the caisson foundation, but in order to provide an effective seal at the cutting edge the whole caisson had to be sunk to a depth of at least 2 feet below the dredged level. An extra depth had also to be allowed under the cutting edge for space in which to cut out the projecting toe. It was decided, therefore, that a revised design should take into account a total extra depth at the toe of 3 feet 6 inches and, owing to undercutting, a net increase in the width of 3 feet. Calculations with these revised figures showed that the toe pressure would not exceed the maximum of 4 tons per square foot.

Consideration was also given to possible failure through sliding and, with conservative values for cohesion and angle of internal friction, the factor of safety lay between 2·1 and 3·6

The stabilities and ground pressure under the three land caissons were all determined in the usual manner; they call for no special comment except that extra calculations were required for two caissons which are non-symmetrical in plan.

Reinforced-Concrete Caissons

After the correct combinations of maximum external loadings on the various units had been ascertained, the details of design for the various caisson units followed normal principles. A tensile stress of 18,000 lb. per square inch in the steel reinforcement was used for the design calculations, and in certain members, the compressive stress in the concrete reached a maximum value of 995 lb. per square inch.

Conflicting requirements of the floated caissons were a minimum weight for launching and floating, combined with a strength adequate to withstand the varied stresses imposed by the operations of constructing, travelling, launching, floating, positioning or grounding, ballasting, and (when the final position in the structure had been reached) filling with sand or concrete.

The problem of weight did not enter into consideration for the design of the three land caissons, and large factors of safety were used to withstand the arduous duty of sinking through a thick bed of boulder clay to the shale and sandstone foundations. Resulting pressures, from the maximum and minimum external and internal forces, were calculated by considering the caisson sunk at full depth with the following assumptions :—

Internal pressures
  (*a*) Air pressure equivalent to level of high water.
  (*b*) Air pressure reduced to 40 per cent of maximum.

External pressures
  (*a*) Dry ground with no water pressure.
  (*b*) Full ground pressure plus water pressure to high-water level.

Walls of the working chamber were designed as cantilevers with a length equal to the height of the working chamber, and the roof was considered as a slab spanning in two directions. Reinforcement in the front wall was calculated to withstand the water pressure at high water plus a load of 5 tons per linear foot acting on any fender.

Many alternative loadings for the ground caissons were produced by the various stages during the operations of floating, grounding, sinking, and final filling with mass concrete. These had all to be considered and calculated to determine the most unfavourable combination which might act on the walls and roof of the working chamber.

The walls of the upper compartments caused no difficulty. For the purposes of calculations the caisson was considered to be grounded, with high-water pressure outside the compartment and atmospheric pressure on the inside. The walls were designed as slabs, continuous over the intermediate walls, with the water pressure causing combined bending and compression. Three reductions in wall thickness are equivalent to the reductions in hydrostatic pressure.

Maximum forces acting in opposite directions, before and after the application of compressed-air, occur in the walls and roof of the working chamber when the caisson is considered as grounded. The walls of the working chamber were designed as cantilevers to withstand the maximum unequal outward and inward forces caused by :

(*a*) Full air pressure inside with low-water pressure outside.
(*b*) Internal air pressure reduced by one-third with high-water pressure outside.

Calculations for stresses in the roof of the working chamber were complicated and took into account the bending moment caused by the cantilever action of the lower walls in addition to the various vertical and horizontal forces of compressed air, water pressure, and ballast in the upper compartments. It was treated as a slab supported around the edges by a beam, with continuity of adjacent panels over the intermediate walls.

Pre-stressing, by post-tensioning high-tensile steel cables according to the Freyssinet method, was introduced into the design of the ground caissons to act as longitudinal reinforcement for the long thin walls of the working chamber which acted as beams during the grounding and sinking operations. It will be realized that considerable accidental bending moments can be produced if the cutting edge encounters boulders, or rock projections, and the value and direction of the moment, sagging or hogging, will depend on the positions of the points of bearing.

Ordinary reinforcement would not have been satisfactory at the toe because the wall was thin (9 inches) and, in addition, post-tensioning ensured the elimination of hair cracks in these important members. In the larger ground caissons the 45-foot-long walls each contain two groups of three cables. One group is positioned as close to the cutting edge as is practicable, and the second group is centred 7 feet above. Each cable is calculated for a load of 20 tons, giving a total resisting moment of 840 tons feet in either direction.

The back caissons were designed as simply supported beams, to withstand the horizontal and vertical pressures exerted by the filling and superimposed loading. Flotation stresses were also taken into account.

All top caissons have a 5-foot seating on the adjacent ground caissons, and they were considered to be simply supported between the mid-points of bearing. The bottom slabs, beams, and outer walls were designed to withstand an external water pressure of 12 feet 9 inches, equivalent to the height of high water with the caisson resting on the ground caissons. The back wall had, in addition, to be designed against the external thrust of the backfilling. Forces acting inside, on the bottom slab, included the vertical loads from the filling, sand or concrete, plus any additions which might be transmitted to the filling by superimposed loading on the deck slab. The two main beams supported the correct proportions of weight from the lower slab and, in addition, a dead load equivalent to 1 cwt per square foot in the conveyor subway, as well as 76-ton point loads from each of the four bogies of a travelling grain intake tower.

CONSTRUCTION

Dredging

While work proceeded on the extensive temporary works ashore preliminary to the building of caissons (see Figs 4, Plate 1), the main dredging programme was put in hand, and in the early stages very good progress was made. A large bucket-ladder dredger fed into dumb, bottom-door hoppers which were towed about 2 miles to a special dumping area in the Firth of Forth. Large boulders in the boulder clay were shortly encountered and, despite the wide experience of the dredger crew, caused numerous breakdowns and delays. In four or five instances explosives had to be placed by divers to break up large boulders embedded in the clay, which it had been found impossible to move by other methods. The same dredger was able to remove all the overlying shale and sandstone, except in one corner near the land caissons, where divers were engaged to drill and blast the top sandstone layers. An approach channel to the new berths was dredged, connecting to the previously dredged portion of the bay, at the same depth of 18 feet below low water of ordinary spring tides, which is equal to 34 feet at high-water springs.

The bottoms of the dredged areas were finished off smooth and the sides, where in boulder clay, were stepped at an approximate slope of 1 : 1. The overlying sand or silt was taken back at the flat angle of 10 degrees to the horizontal. The site of the new quay walls was dredged at the same time as the adjacent berths, allowing a minimum horizontal clearance of 5 feet between the back of the quay walls and the toe of the slope.

The total dredging amounted to about 250,000 cubic yards, half of which was in hard materials.

Land Caissons

Concurrently with the dredging operations, work was started ashore to prepare the site for the most westerly land caisson. One corner of the area protruded on to the foreshore and the first procedure was to build a triangular cofferdam of large pre-cast hollow concrete blocks. The blocks and space behind, were filled in to about 1 foot above high-water level with fill obtained from the adjacent bank.

On this levelled area the steel cutting edge, measuring 69 feet by 42 feet, was erected on a shallow concrete kerbing, followed by the shuttering for the haunch and roof of the 7-foot-high working chamber. The outside faces of the walls were set to a batter for a height of 11 feet above the cutting edge, to overcome an anticipated difficulty when sinking through tough boulder clay into shale. When the walls above the roof of the working chamber had progressed a sufficient height for clearance above the outside ground, the supports under the roof of the working chamber were removed. Owing to the soft and uneven nature of the clay filling underneath, the caisson, then weighing over 1,200 tons, tilted and sank unevenly into the ground. The caisson was righted by careful sinking, and excavation continued for some days until water from the high tides began to seep through the surrounding earth and under the cutting edge. Air locks were then attached to the two air shafts and sinking proceeded continuously through soft materials, with two squads of eighteen men each working 11-hour shifts. The outside batter gave a lateral freedom of movement of 12 inches in each direction, and it required constant vigilance to maintain correct alignment. Boulder clay was encountered at the expected level and sinking was slowed up very considerably by the large number of boulders. Pneumatic clay-spades with high-pressure air were employed to excavate in both boulder clay and the soft shale encountered in the later stages.

The outside batter to the lower walls obviated any need for kentledge and in the soft ground it was difficult to build the walls on top with sufficient speed to keep pace with the sinking. The outside batter on the walls proved troublesome by allowing "blows" through the loose materials, which fell in and failed to form an effective seal with the outside surfaces of the walls. Sinking proceeded until a suitable foundation was established with the cutting edge at a depth of 35 feet below low-water level. The caisson was then levelled, soft spots removed, and the working chamber filled with concrete and sealed with grout under pressure.

The reinforced-concrete walls of the upper compartments were 10 feet thick at the back and 4 feet thick elsewhere, and were built up in lifts of 3 feet 3 inches. Intermediate walls acted as stiffeners between the front and back walls and formed three voids 17 feet 6 inches square, which were filled with water ballast up to mid-tide level before the deck was concreted. The deck was of 4-foot-thick reinforced concrete and, when completed, the caisson weighed 7,900 tons, excluding the water ballast.

The maximum air pressure reached in the working chamber was about 24 lb. per square inch at the lowest depth. That did not cause any discomfort to the men and the time of decompression was about 6 minutes.

The other two land caissons were of similar design, except for the corners cut off by the re-entrant angle which did not cause any special difficulty in sinking. To cause minimum disturbance to the ground, the two outer caissons were sunk and sealed before the central one was started (*Fig. 5*). Gaps of 4 feet were left between each caisson and were filled in near the front by driving overlapping steel sheet-piling, one pile having previously been fixed to the side of each caisson before sinking commenced. Above low-water level the gaps were filled with mass concrete, and drainage from the back of the quay was provided through the same space by pipes laid during the concreting operations.

Ground Caissons

Building.—Extensive office and outside works were required before the actual building of the first ground caisson could be commenced.

Calculations had to be made and drawings completed before orders could be placed for the various items of specialized equipment necessary for the launching, floating, and positioning of the caissons. At the site, after the methods of building and launching had been carefully considered and approved, the preparation of the concrete building bay and launchway consumed much labour, material, and time. The anticipated delay was aggravated by an unforeseen occurrence when, owing to the restriction on capital expenditure, H.M. Government issued an order to stop the work. About 3 months later that order was rescinded but, by that time, the construction programme had been completely disrupted.

From cross-sections taken of the bank and foreshore, it was found that a vertical curve of 380-foot radius could be drawn between a horizontal line at the top of the building bay and the point where extreme low water intersected the foreshore. It was decided to adopt this radius as the top profile for the launchway walls and so permit all the foundation work to be completed in the dry between tides. At the lower end where the beach was soft, the foundation was supported on stump piles, and short walls of steel sheet-piles were driven to prevent spread.

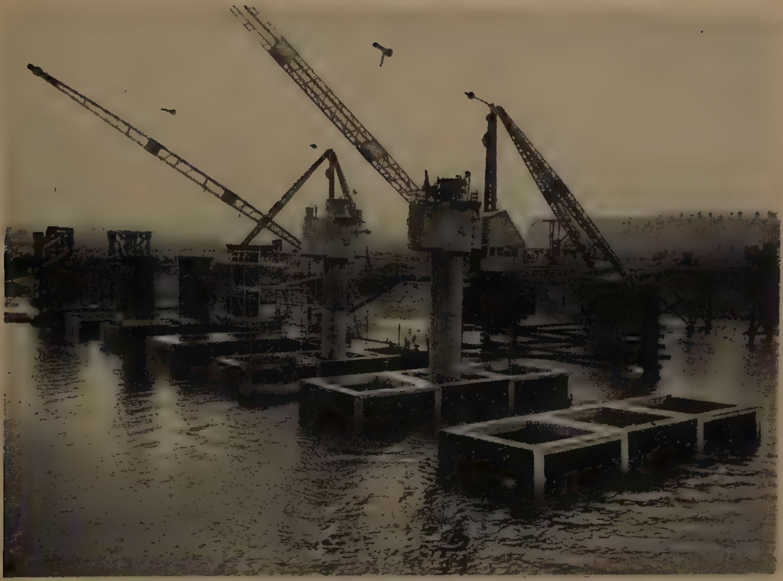
The main dredging had been substantially completed before the first steel launching carriages and frames were delivered, but, as soon as they arrived, they were assembled at the ground-caisson building bay. The strongly reinforced shells of the ground caissons were cast, lying on their sides, over two specially constructed carriages which supported the soffit shuttering. By casting in this position their draught was reduced, enabling them to be floated off the launchway, using the 16-foot rise of tide, without dredging the shallow foreshore. Incidentally, the side casting made it possible to eliminate about 3,000 square yards of shuttering.

The building bay was large enough to accommodate three caissons at one time in different stages of construction. As soon as the forward caisson had been completed and launched, the two others, still under construction, were moved forward. The free carriages were then recovered at low tide from the foot of the launchway, placed at the back of the bay, and prepared for building the next caisson.

The actual construction of the caissons at the building bays calls for no special comment, except that the shuttering had to be accurately set and rigidly secured. All the concrete was of good quality, nominal 1 : 1⅓ : 2⅔ mix, with ¾-inch graded aggregate, and vibrated with compressed-air-driven immersion vibrators. The roof of the working chamber incorporated a permanent 2-foot length of air shaft and three pipes—two for the supply of air at high and low pressure, respectively, whilst the third (3 inches in diameter) was used as a " snorer " to eject any surplus water which might collect in the working chamber.

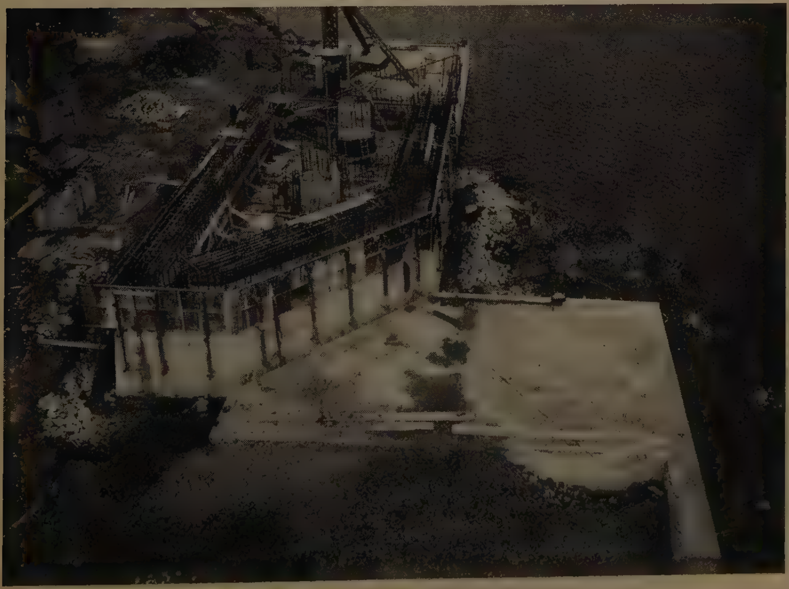
Pre-stressing.—Tensioning of the high-tensile-steel cables by the Freyssinet method was undertaken as soon as the final pour of concrete was

Fig. 2



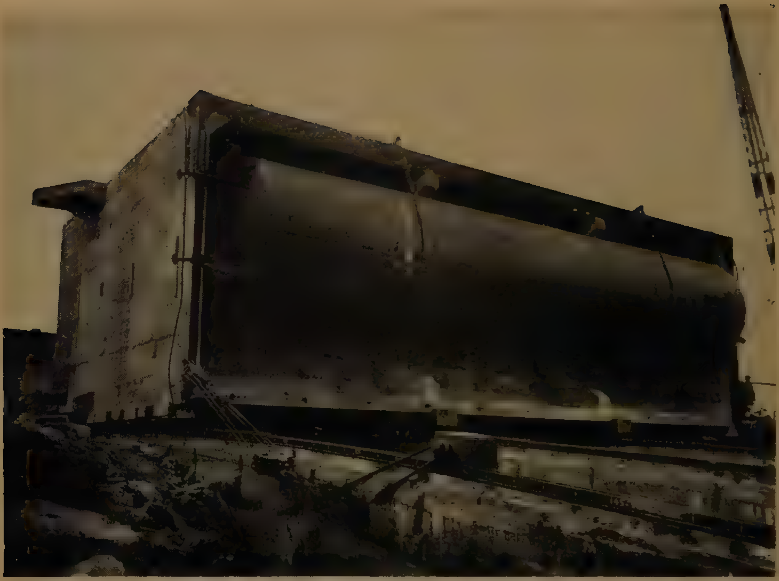
Various Construction Stages of Ground Caissons. View at Extreme Low Water

Fig. 5



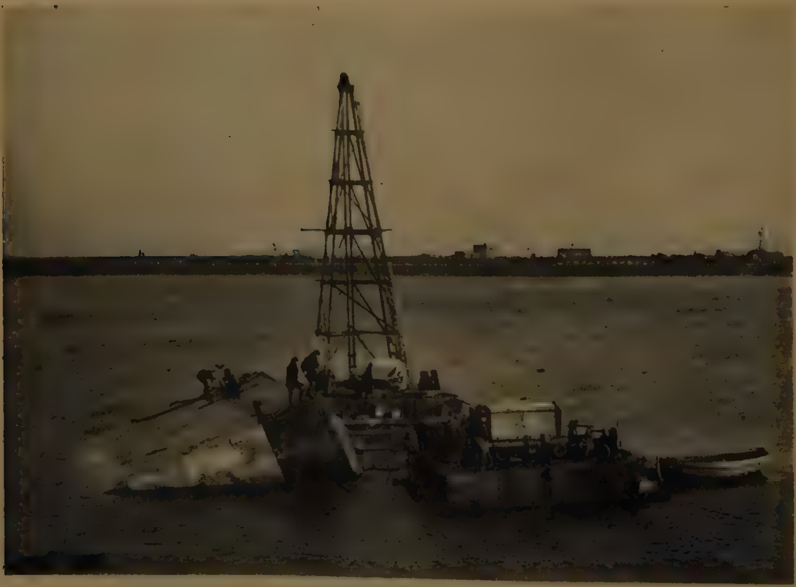
Land Caissons—Two Outer Caissons Completed

Fig. 6



Ground Caisson—Travelling to Top of Launchway

Fig. 7



Ground Caisson at Floating Angle

known to have reached a compressive strength of 4,000 lb. per square inch.

The high-tensile wire, 90–100 tons per square inch, arrived on the site in small-diameter coils and had first to be straightened by passing through a spinner-type straightening machine. Each cable comprised twelve 0·2-inch-diameter high-tensile wires grouped round a central $\frac{1}{2}$-inch-diameter spiral, which was made out of ordinary 16-gauge annealed wire. The outer diameter of the cable was approximately 1 inch, and the wires were bound tightly in position at 12-inch centres along its length. They were then wrapped inside two overlapping sheaths of 24-gauge strip, again bound tightly, but at about 4-inch centres, to prevent concrete gaining access to the cable. The completed cables were placed at the same time as the ordinary reinforcement, with the ends passing through a Freyssinet female anchoring-cone. These anchoring cones were set in a 2-inch recess, and at the inner end a bearing plate with an area of 30 square inches transmitted the thrust to the concrete.

When ready for stressing, the wires at one end were securely fixed by driving home the shaped male cone and, at the other end, after inserting a male cone, the twelve wires were distributed and wedged into a double-action Freyssinet jack which was connected to two Tangye hydraulic pumps. Tension was imparted to all twelve wires at the same time by the extension of the main jack head until a pressure of 4,000 lb. per square inch was registered on the pump gauge. This imparted a tension of 60 tons per square inch to the high-tensile wires, or 24 tons per cable. The observed extension on the cables was $2\frac{7}{16}$ inches. When this tension had been obtained the second pump was operated until that gauge reading was 5,000 lb. per square inch. This pressure was transmitted to the internal blocking jack, which acted on the male cone with a thrust equivalent to 15 tons and effectively secured the wires between the serrated edges of the male and female cones. The jack was then released and the hollow cable filled with a neat cement grout under pressure. This completed the operation and after the group of three cables had been tensioned the ends of the wires were burned off and the recess filled with mortar.

Launching and Positioning.—When the caisson was ready for launching it was pulled forward to the vertical curve of the launchway, where a large steel buoyancy tank was attached by strops fixed at one end to a bolt, specially cast in the roof of the working chamber, and carried under and round the tank to the cutting edge (see *Fig. 6*). Three watertight covers, with manhole openings, were also fixed at the top of the caisson to cover the upper compartments, and a special watertight cover was fixed over the lower end of the air shaft.

All travelling motions were made over trains of $2\frac{1}{2}$-inch-diameter cast-steel balls held in spacer plates and rolling in the grooves between two 95-lb. bull-head rails laid on their sides. The lower rail was welded to a mild-steel base-plate, which was permanently attached by coach screws to timber beams on top of the launchway walls, whilst the upper rail formed part of the launching carriage. The lowering was accomplished by two specially designed 7-ton hand-winches, working through purchase blocks with a 8 : 1 ratio and capable of a maximum horizontal travel of 35 feet.

The two travelling and launching carriages were mounted on bogies, four to each carriage, with articulated joints at 17-foot centres to accommodate the curve without imposing any bending strain on the caisson. At the end of the launchway, in the floating-off position, the caisson was inclined at an angle of 18 degrees to the horizontal, with the heavy end, incorporating the steel cutting edge and working chamber, towards the water.

Three stop positions for the launching carriages were arranged at approximately 32-foot centres down the launchway; the first stop position being shown in *Fig. 6*. On reaching a stop position, the weight of the caisson was transferred from the winches to the stops, allowing the tackle between the blocks to be shortened and a longer pair of strops inserted between the front blocks and the bridles at the rear ends of the launching carriages. The winches were synchronized at the first stop position to ensure equal lowering. The time taken was 40 minutes between stops, with a 2-hour pause at each position to arrange and check the lowering tackle for the next stage.

At low tide, before floating off, the caisson was lowered to the foot of the launchway beyond the third stop position and the carriages were anchored by a rope, passed round a pulley in the foundation to prevent lifting by adhesion of the concrete. As the tide rose and buoyancy provided some reduction in weight, the caisson was lowered a further 12 feet on to a cantilever extension of the launchway walls to obtain full benefit from the rise of tide. When the tide rose still further, the two strops securing the buoyancy tank were removed. The tank was thus free and held in position only by the 18-degree forward tilt and its own upward thrust against the arched haunch of the working-chamber wall.

When the tide reached the correct level the caisson floated off gently and, owing to the buoyancy tank, remained floating with the 18-degree forward slope (Fig. 8 (*a*), Plate 2). The correct size and dimensions of the tank had been determined by careful and detailed calculations of vertical forces acting on the combined unit with the resulting balancing moments of gravity and buoyancy. The tank was ordered with a slight surplus buoyancy which could be balanced with water ballast.

The stability of the floating unit was investigated for all stages of the operation and the positions of the centres of buoyancy and gravity relative to the metacentre were calculated.

Towing to position at the dredged berth was performed by two motor launches, one pulling and one restraining, where it was handed over to a positioning frame comprised of two pontoons, firmly braced together, with a removable beam at one side. The caisson was handed over with the top towards the positioning frame, which was near but not over the final site for the ground caisson (see *Fig. 7*).

The next stage was to upright the caisson by flooding the buoyancy tank. This was accomplished through two 2-inch-diameter hoses connected to each end of the tank. When flooding was completed the tank dropped clear to the sea bed and left the caisson floating in the correct upright position with a freeboard of 6 feet 2 inches (Fig. 8 (*b*), Plate 2). The caisson was then entered into the positioning frame and secured by wedges at all four sides. Final positioning was done on a falling tide while the unit was still afloat. The positioning frame was accurately set to line by theodolites, and the movement was controlled by six winches, three mounted on each pontoon.

Valves in the caisson walls were then opened and the voids flooded, causing the caisson cutting-edge to ground on the sea bed. A tolerance of 3 inches in each direction was allowed for positioning the ground caissons, and final settings were all within this limit. The buoyancy tank was later recovered by blowing out the water with compressed-air and so restoring the buoyancy.

After the caisson had been grounded, the air shaft was extended to above high water and the air lock fixed on top. Compressed-air was then applied to the air lock and working chamber simultaneously in order to displace the water. The water in the shaft was blown into the adjacent upper compartment through a small vent-hole, specially constructed at the foot of the shaft (Fig. 8 (*c*), Plate 2). This enabled a man to descend, plug the vent-hole, detach the cover at the foot of the air shaft, and gain access to the working chamber.

All soft silt in the working chamber above the dredged level was removed by bucket through the muck lock and the foundation stratum was closely inspected. When a suitable foundation had been attained, the working chamber was concreted in the usual manner and sealed with cement grout held under pressure for about 2 days (Fig. 8 (*d*), Plate 2).

All excavating and concreting operations in the working chamber, which was 5 feet 6 inches in height, were carried out under compressed-air by two gangs of nine men, each working 11-hour shifts. The maximum pressure at high tide was about 22 lb. per square inch. A supply of high-pressure air was available to operate the pneumatic tools.

The air lock and top portion of air shaft were then removed and watertight trunkings, extending above high water, were fixed above the three voids (Fig. 8 (*e*), Plate 2). The voids were pumped out one at a time and filled with a weak mass concrete, thus providing a solid pier of concrete as the substructure. At low water the top levels of the projecting piers were ascertained, and an outer course of pre-cast blocks, backed with mass concrete, was built to the correct level for seating the top caissons (see *Fig. 2* and Figs 3, Plate 1).

Rock Bank

As each ground caisson was completed, the placing of the intermediate rock bank was commenced. The rock, ½–2 cwt size, was brought to the site by motor tippers in 5-ton loads and discharged into skips designed to take a full motor load. These skips were handled, from tip to rock bank, by two 10-ton derricks mounted on timber dolphins behind the quay wall (*Fig. 2*, facing p. 68). The bank of large rock was completed in this manner up to the level of the underside of the back caissons, soundings being taken from a raft to ensure a correct profile.

The 10-ton derricks were built, complete with jibs, on three pontoons near the site of the ground-caisson launchway, floated out at high water, and positioned over dolphins on a falling tide. When the tide fell below the tops of the dolphins, the two back-leg pontoons were each tied down to the dolphins by four 1⅜-inch-diameter bolts, and the pontoons were filled with water ballast. After one section of the wall had been completed, ballast and back weights were removed, and the derricks were floated over the next set of dolphins. The displacement of each derrick was about 50 tons and the maximum draught of the pontoons when brought to a level trim was 4 feet 8 inches ; *Fig. 2* shows the derricks on dolphins.

Back Caissons

The back caissons were constructed on a specially prepared concrete platform and were in the form of hollow boxes. When completed, they were travelled by special carriages moving over trains of steel balls, and placed for launching, on the same carriage and launchway as the top caissons.

Buoyancy for flotation was imparted by an assembly of six steel buoyancy tanks, built to enter the space between the ground caissons. Once the back caisson had been positioned on the cantilever bracket of the ground caisson, the buoyancy tanks were released. Water ballast in the tanks, used to position the caisson, was blown out by compressed-air and the whole assembly then floated clear over the top of the caisson. Principal dimensions were as follows :—

Caisson	Size	Launching Weight : tons	Draught
Large	28′-0″ × 6′-0″ × 10′-6″	44·7	7′-7″
Small	23′-9″ × 6′-0″ × 7′-0″	24·8	6′-9″

Any make-up required, owing to sinking of the ground caisson, was accomplished before launching by bolting pre-cast blocks of the correct height to the underside of the projecting supports.

The screen of small broken stone was placed over the reverse slope of the rock bank and against the back side of the back caisson (*Fig. 9*). The hollow compartments in the back caissons were then filled with concrete which was allowed to penetrate and form a seal with the top of the rock bank below.

Top Caissons

The top caissons, of two different sizes and types, were constructed over a building bay sited at right angles to the launchway and of sufficient length to accommodate three caissons under construction at the one time. The building bay comprised four parallel concrete walls, surmounted by 95-lb. bullhead rails laid flat and firmly attached to timber beams. The main outer walls were 27 feet apart and the inner walls, at 15-foot centres, were lower and supported the travelling carriage.

Concreting calls for no special comment and the building of even the larger type could be completed in a period of 4 weeks. The reinforcement skeletons for the beams were built in advance on the ground alongside and hoisted into position by a derrick. All concrete was vibrated by immersion-type vibrators and no pre-stressing reinforcement was required.

In order to prevent any distortion the soffit shuttering was firmly supported on the travelling carriage and on the outer walls, whilst the extremities of the larger caissons were blocked up on special foundations which incorporated sand jacks. Concreting continued in the one position until the new reinforced-concrete beams had developed sufficient strength to withstand the bending moment for a clear span of 27 feet. When that stage had been reached the outer supports were removed and the whole unit was pulled forward over a point where the carriage ran down an incline and dropped clear, thus transferring the whole weight to beams travelling over the outer walls. That released the carriage and soffit shutters, which were then set up at the back of the building bay for the construction of the next unit while concreting of the previous caisson was being completed. As with the ground caissons, all travelling movements were accomplished on continuous trains of 2½-inch-diameter cast-steel balls, rolling in the grooves between two 95-lb. bullhead rails placed on their sides.

The launchway walls were constructed to a slope of 1 in 6½, this being determined by the distance to low-water mark. At this slope it was possible, provided that the long side of the caisson was presented to the water, to obtain sufficient depth of water to float the larger caissons without the requirement of a buoyancy tank.

A launching carriage of triangular shape, to suit the slope of the launchway, was controlled by two 7-ton synchronized hand winches and, before the transfer of a caisson, the carriage was housed near the top of the launchway. The caisson was traversed over the carriage and, when in the correct position, there was about ½ inch clearance between the underside of the caisson and the top of the carriage. The launching carriage was then pulled against the slope by the hand winches, assisted on the other side by the thrust from three heavy-duty jacks, and the resulting wedging action of the carriage relieved the weight from the transverse travelling beams. These were detached and, after dismantling a structural-steelwork gantry on the water side, the carriage was free to be lowered under control of the winches. Lowering to flotation position was accomplished with three stop positions in a manner similar to the ground caissons, except that, in this case, the units remained in a horizontal position on the carriage throughout the course of travel.

The principal dimensions were as follows :—

Caisson	Size	Launching weight : tons	Draught	Freeboard
Large	45′-0″ × 33′-3″ × 14′-8″	383	8′-11″	5′-9″
Small	45′-0″ × 29′-0″ × 13′-0″	297	7′-8″	5′-4″

The larger caissons, for flotation, required temporary watertight timber cofferdams to cover the subway openings as these came below water level.

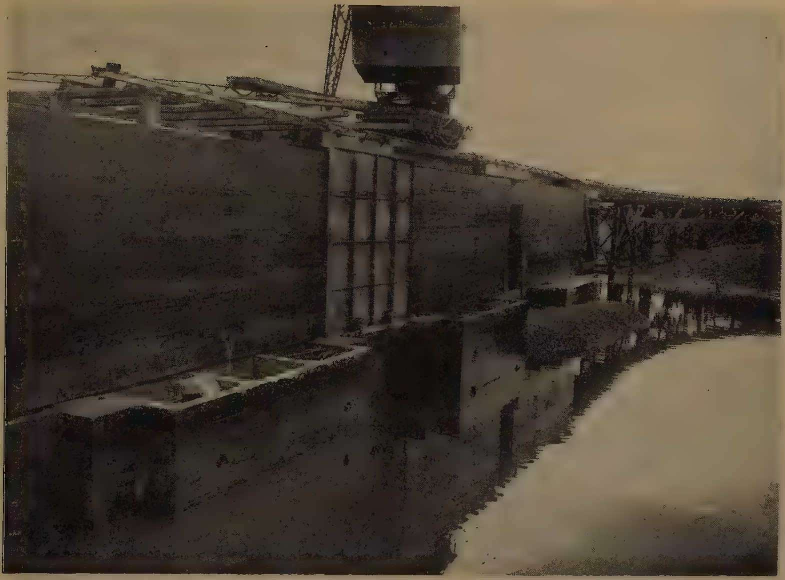
At high water the top caissons were floated over the prepared ground caissons and positioned gently, as the tide fell, by admitting water ballast to the inner compartments. The final line and position were accurately determined by marker guides attached to the previously set top and ground caissons. Sufficient water ballast was admitted through the flooding valves to prevent further movement until such time as the sand filling could be placed. Surplus water was then pumped out before the front compartments were filled with concrete.

Corner Caisson

The caisson at the apex of the triangle, forming the junction between the 500-foot and 424-foot berths, consisted of a single reinforced-concrete unit, measuring 52 feet 6 inches by 48 feet 6 inches in plan and 47 feet 6 inches in height. The weight and size of a portion, with sufficient height for buoyancy, would have been too great for the existing launchways and, in addition, the shallow depth of water would have required special buoyancy tanks or extra dredging.

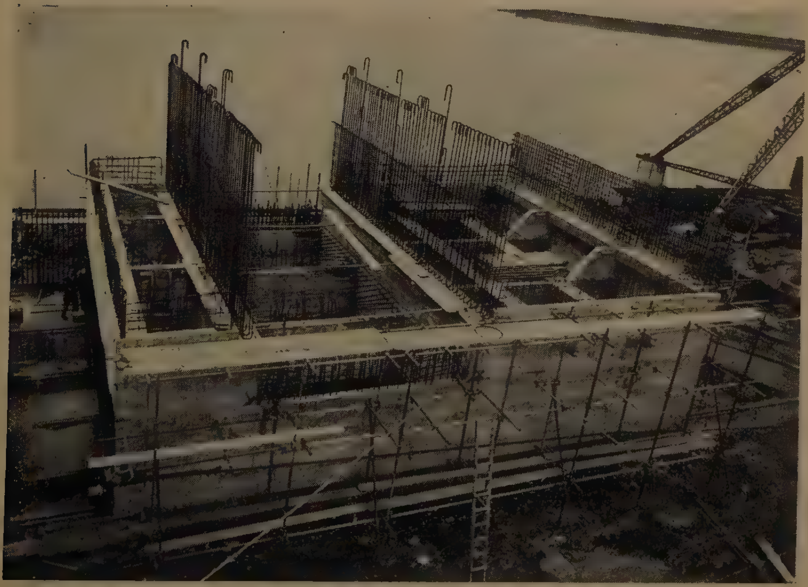
It was decided, therefore, to build a steel shoe—incorporating a working chamber 5 feet 6 inches high—on shore, to launch it, and complete the concrete superstructure over this floating base before setting in its final position. The 14-foot-3-inch-high steel shoe was built over two launching beams, which overhung the edge of the bank. Plating, for the outside skin, roof, and haunches of the working chamber, was all ¼-inch thick, and the main structure comprised twelve intersecting plate web girders, 5 feet 6 inches in height, fixed in position above the roof of the working

Fig. 9



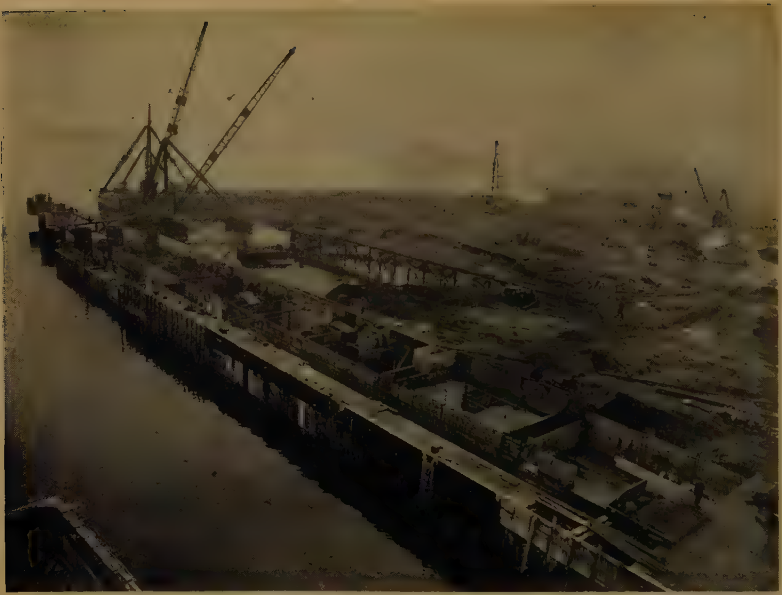
Back of Wall at 424-foot Quay. View at Extreme Low Water

Fig. 10



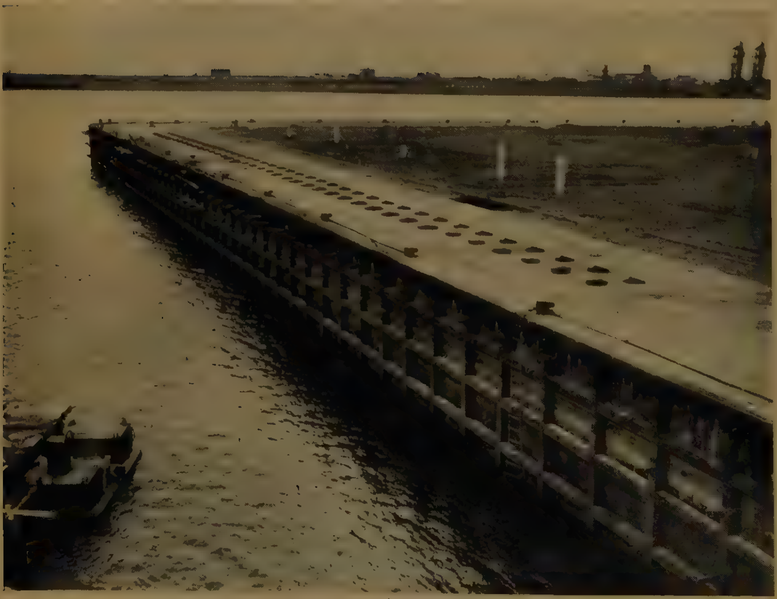
Top Caisson—Large Type under Construction

Fig. 13



View of 500-foot Quay

Fig. 14



View of Part of 500-foot Quay, 8 June, 1950 chamber. Joints of the skin plates were butt-welded, and a watertight test was made before launching. The launching weight amounted to 105 tons, including 19 tons of reinforcement which had to be placed before the outer skin plate was fixed.

When completed the unit was tilted to a slope of 1 in 3 and, after the launching beams and rails had been extended down to the foreshore, launched at high tide; it was then moored in front of the three land caissons at the completed corner of the quays.

When launched, the steel shoe had a freeboard of 8 feet 3 inches, but by the time the 12-inch-thick concrete walls had been brought to the same height the freeboard had been reduced to 2 feet 11 inches. Concreting of the 12-inch external walls, which were strengthened by four horizontal ribs, proceeded in lifts up to a total height of 50 feet 2 inches, when the total weight was 2,580 tons and the draught 42 feet 9 inches. The step-in, shown in *Figs 12*, was made on two adjacent sides to provide landings for the top caissons. Great care had to be taken throughout the construction to maintain a level trim and sufficient freeboard.

This large floating unit was moved into its final position by winches mounted on the caisson itself, and on touching bottom the internal chamber was flooded to a depth of 10 feet 6 inches to produce zero buoyancy. As soon as possible thereafter the 5-foot-thick mass-concrete internal wall was poured in advance of the sand filling. When sufficient weight had been added, compressed-air was applied to the working chamber and the foundation completed in the usual manner.

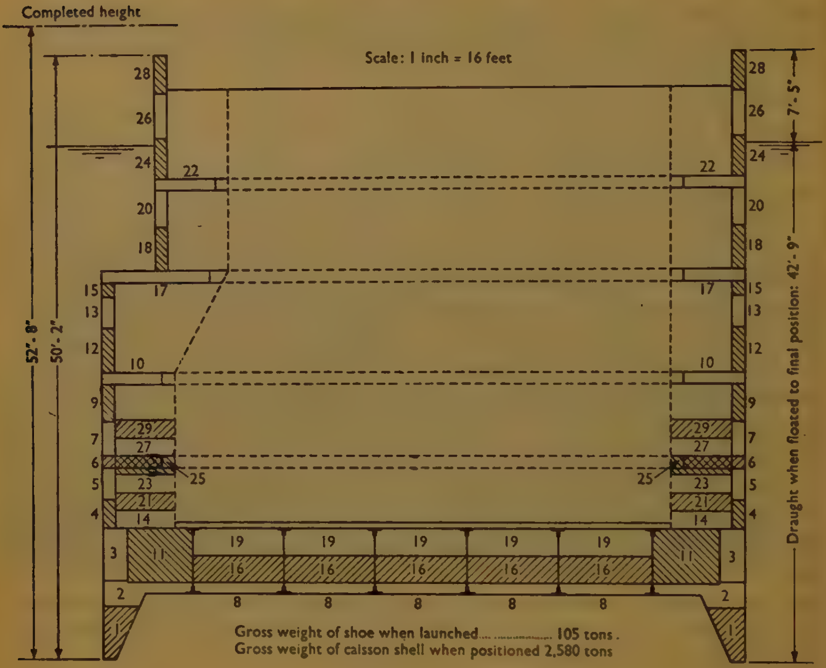
Superstructure and Decking

The only portions of quay wall below high-water level were the gaps between the top caissons, and this concrete work was all performed in the dry between tides. Reinforcing bars, which had been left projecting from the ends of the top caissons, were straightened and welded together before the concrete was poured. In the same gaps vertical reinforcement for the mooring bollards, consisting of old rails, was carried right down into pockets left in the front portion of the ground caissons.

The 4-foot height of front wall, above the top caissons, was approximately 7 feet 6 inches thick and was reinforced to withstand the impacts from shipping. Concreting of the conveyor-subway floor-slab was carried out in the dry, the end cofferdams being left in position until the front and back walls of the gap had been completed above high water.

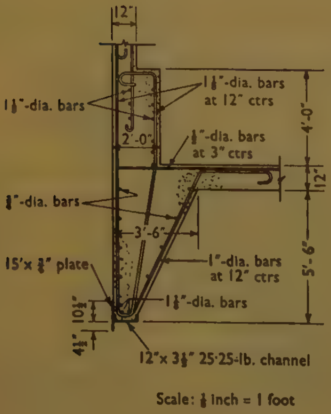
The deck slab was 10 inches thick, reinforced top and bottom with layers of “ Square-grip ” fabric, and with expansion joints at intervals in both directions. Cast-iron spouts for grain and dust and patent “ Glasscrete ” pavement lights were concreted-in with the roof of the conveyor subway. The whole area was surfaced with a 2-inch thickness of “ Lithocrete ” mastic asphalt, laid in two layers, and with a non-skid crimped finish.

Figs 12



Scale: 1 inch = 16 feet

Gross weight of shoe when launched.................. 105 tons.
Gross weight of caisson shell when positioned 2,580 tons



Scale: ⅛ inch = 1 foot

Pour number	Height of caisson	Freeboard	Concrete: cu. yds
At launch	14'- 3"	8'- 3"	-
1	"	-	69
2	"	-	66
3	"	-	56
4	"	2'- 11"	18
5	16'- 0"	4'- 2"	20
6	17'- 3"	6'- 6"	48
7	20'- 0"	5'- 3"	20
8	"	7'- 3"	49
9	23'- 0"	7'- 4"	22
10	24'- 3"	4'- 4"	48
11	"	-	125
12	27'- 3"	-	22
13	30' -3"	9'- 2"	22
14	"	7'- 8"	60
15	31'- 6"	8'- 8"	9
16	"	4'- 8"	155
17	32'- 9"	"	48
18	35'- 11"	7'- 4"	21
19	"	5'- 0"	90
20	39'- 2"	7'- 8"	22
21	"	6'- 11"	30
22	40'- 5"	"	37
23	"	5'- 1"	60
24	43'- 8"	7'- 5"	20
25	"	6'- 6"	30
26	46'- 11"	8'- 10"	20
27	"	6'- 5"	85
28	50'- 2"	8'- 9"	20
29	"	7'- 5"	43
		Total concrete	1,335

Corner Caisson, Concreting Table, and Details of Working Chamber

Concrete and Concreting

Four qualities of concrete with the following proportions were used :—

Quality " B ", 1 : 1⅓ : 2⅔ (coarse aggregate graded ¾–⅛ inch)
,, " C ", 1 : 1½ : 3 (,, ,, ,, ,, ,, ,,)
,, " D ", 1 : 2 : 4 (,, ,, ,, 1½–⅛ ,,)
,, " F ", 1 : 4 : 8 (,, ,, ,, ,, ,, ,,)

The cement was proportioned by weight and the aggregates by volume, with provision for an increase in cement if the voidage in the aggregates exceeded a certain percentage. For important positions the proportioning was converted to weight values and the aggregates were accurately gauged through a Blaw-Knox " Agrimeter."

Quality " B " concrete was used for the reinforced-concrete shells of the corner and ground caissons, for underpinning foundations, and for sealing the working chambers of the foundation caissons. A crushing strength of 4,000 lb. per square inch was required and this strength was normally exceeded between 7 and 14 days. All other reinforced concrete was of qualities " C " or " D," the latter generally being employed where the dimensions of the wall or slab and the spacing of the reinforcement permitted free working and flow of concrete with the larger-sized aggregate. For both these qualities the minimum compressive strength required at 14 days was 3,000 lb. per square inch. The weakest concrete, quality " F," was principally used for the mass-concrete filling in the upper compartments of the ground caissons.

A British Portland cement was used for all reinforced-concrete work, although on occasion, when a high early strength was desired for some specific purpose, a rapid hardening Portland cement was permitted. At one period during the contract a shortage of home supplies compelled the introduction of a German cement, but despite satisfactory tests, its use was limited to mass-concrete work where the slower setting time was beneficial.

Sand was transported considerable distances by road from two selected pits. It was all washed twice and the sieve analysis showed that the grading lay within the limits prescribed by B.S. 822, 1944, for a Class " A " sand. Coarse aggregate was a good-quality crushed whinstone, free from dust, graded from 1½ inch down to ⅛ inch, or from ¾ inch down to ⅛ inch.

Three concrete-batching plants, of either ¾-cubic-yard or ½-cubic-yard capacity, were erected at positions to suit the principal concrete pours, and one ½-cubic-yard portable mixer was used for temporary works and, when required, as an auxiliary to augment the concrete supply to any particular point. The mixers at the fixed batching plants were fed from overhead hoppers filled by grabs from the various stock piles of aggregates. All cement was delivered in 1-cwt bags to cement sheds erected beside the plants and transported to the mixer hoppers by conveyors.

An accurate control of the mixing water proved troublesome, owing to the variable moisture content in the exposed heaps of sand but, with experience, the mixer attendants were able to maintain a given consistency with considerable accuracy. A pycnometer jar was used to determine the moisture content in the sand, but the results, although accurate, could prove misleading when translated from the sample to the actual stock pile. Slump tests were taken from time to time as a check on visual control. A consistent dry mix could be produced and maintained, but a limiting factor for the dryness was the physical difficulty of discharging the mixture from the concrete skip after transportation. The water/cement ratio varied from 0·43 to 0·56, depending on the position of pour and quality of concrete required.

Transport between mixer and position of pour was generally by trains of skips, of side-gate-discharge or bottom-dump types, propelled by diesel locomotive on narrow-gauge track (see Figs 4, Plate 1). In certain positions, at the caisson building bays for example, the whole operation lay within the radius of a derrick. A concrete pump, with a 4½-inch-internal-diameter delivery pipe, was introduced in the later stages of the work and proved extremely effective for the in-situ concrete at the 500-foot berth. An interesting attempt was made to use the pump for sealing and concreting the working chamber of the large corner caisson. Since the joint where the delivery pipe entered the air shaft was improvised, the experiment was only partially successful, but it is considered that, with proper preparation, this method could be used in large caissons to expedite and reduce the cost of this expensive operation.

Test cubes, in groups of three or six, were cast at frequent intervals from the mixed concrete, and crushed at the site in a testing machine which incorporated a 100-ton hydraulic jack as the main component. The age of the specimens, when broken, ranged from 3 days to 3 months, depending on the position of the pour and the nature or extent of the information required. Post-tensioning was always regulated by the compressive value of the test cubes taken from the actual pour containing the groups of cables.

The concrete for all types of caissons constructed at the building bays was consolidated by immersion-type vibrators operated by compressed-air. These vibrators were most effective in producing a dense watertight concrete with a good surface finish, and permitted the use of a much drier mix than would have been possible with other methods.

The whole of the reinforced-concrete surfaces exposed to sea water, above the level of low water, were treated with three coats of "Melanoid" bituminous paint.

Fenders

A solid system of timber fendering was provided and consisted of backing pieces, made out of 12-inch-by-12-inch creosoted Douglas fir, arranged in a regular vertical and horizontal pattern to suit the spacing of the ground caissons. These timbers were attached to the quay-wall superstructure by rag-bolts, and to each other by bolts and angle cleats. Vertical 12-inch-by-6-inch hardwood rubbing pieces, of Australian brush-box, were spiked to each vertical and secured near the top with 4-inch-by-½-inch bent metal straps, recessed into the face of the hardwood and bolted through the backing piece. Horizontal struts, 12 inches by 4 inches in section, were spiked to the horizontal backing pieces between the vertical rubbing pieces.

The cope was a 14-inch-by-14-inch timber of creosoted Douglas fir with the outer corner rounded to a 3-inch radius. This nose was protected with a ¼-inch-thick rounded steel plate, bedded in red lead putty, and fixed in two directions. The cope was held to the reinforced-concrete structure with bolts. To facilitate replacement of this timber, which is liable to damage, the bolts were inserted head outwards with the nut contained in a small pocket filled with a weak mortar.

Temporary Land Connexions

A temporary connexion to the land was required at each end of the new quay walls to provide access roads and to support and protect the new reclamation at the corners. At the east end a wall, of No. 5-section Larssen steel sheet-piles, was driven into the boulder-clay stratum in a line curved to meet the old bank. The heads of the piles, finished at 6 feet above low-water level, were tied back with 3-inch-diameter tie-rods into an anchor wall of short sheet piles. Above the piles the bank rose at a slope of 1 in 1½ and was faced with hand-set stone pitching.

The connecting wall at the west end, also composed of No. 5-section Larssen interlocking steel sheet-piles, filled the gap between the back of the westmost land caisson and the old bank at that end. The levels of the piles were finished at deck level with a mild-steel capping plate, and the supporting walling, with anchoring 3-inch-diameter tie-rods, was positioned 12 feet below the pile tops.

Reclamation

The triangular water area behind the quay walls was reclaimed with burnt shale, which was readily obtainable from the sites of old collieries in the neighbourhood. Only well-burnt shale of a reddish appearance was accepted. Chemical analyses of the shale were made from time to time. The *p*H value varied from 7·0 to 8·0 and the distilled water showed no tendency to develop acidity, despite a small percentage of sulphuric anhydride in the chemical composition. Compaction and drainage being good, the shale was considered to be a suitable material and not likely to be injurious to the concrete.

Filling proceeded from the bank outwards by means of end-tip motor lorries, and was levelled to just above high-water level by a bulldozer. Thereafter it was brought up in layers, being levelled and consolidated by bulldozer, until the final level was reached. As each portion of quay wall was completed, reclamation followed closely behind towards the outer corner (see *Fig. 13*, facing p. 75) where the soft mud from the sea bed was pushed into a pocket in advance of the heavier shale. This soft material was removed before filling in the corner with good material. Tipping proceeded steadily for a period of 13 months and required a total of 115,500 cubic yards of burnt shale.

Additional Details

Moorings.—Mooring bollards of the ramshead pattern were provided for both berths at the same spacings as the respective ground caissons. They were anchored by rails and reinforcement into the 5-foot-wide gap between the top caissons.

Six mooring posts for storm conditions were provided in addition to the bollards. Each post was designed to take a horizontal pull of 100 tons.

Handrails.—Lengths of protective galvanized handrailing, with short link chains, were provided at the external and internal corners of the quay walls. The stanchions, of forged wrought iron, were collapsible, being tapered to fit into special cast-iron sockets set in the concrete deck, and secured to the socket by a pin passing through a slot in the foot of the stanchion. The stanchions, if in the way of mooring ropes, can thus be laid flat on the deck, but cannot be accidentally detached or lost over the edge of the quay.

Track for intake towers.—The 500-foot berth is equipped over the central 380-foot length with rails, set to a gauge of 16 feet 6 inches, for the travelling pneumatic grain intake towers. Both front and back rails are comprised of double B.S. 75-lb. F.B. rails, with joints staggered, and secured by long rag-bolts to the reinforced-concrete beams underneath. The rails are drained at intervals, and the joints of the front rail are electrically bonded and earthed to the water main.

Services for shipping.—Provision in the quay walls was made for water, electricity, and telephone services to shipping.

British Standard 3-inch screw-down valve hydrants were spaced at intervals of about 60 feet and connected to a 6-inch cast-iron ring main incorporating control valves to isolate any section.

Six-way cable ducts for electric cables have been laid throughout the full lengths of the quays. Pits for junction boxes were built at the end and centre positions of each berth.

A cross duct was laid in the decking for future ship-shore telephone service.

COSTS

The 1,044 linear feet of deep-water quays cost approximately £450 per linear foot. This figure excludes those items extraneous to the quay-wall construction, but does include that portion of dredging in way of the quay walls.

Although the cost may appear high, it should not be assumed that the design was expensive. It should be considered in relation to the conditions pertaining to the site and the special functions required of the quay walls.

ACKNOWLEDGEMENTS

Mr T. A. S. Fortune, O.B.E., M.I.C.E., formerly Chief Engineer to Leith Dock Commission, and Messrs L. G. Mouchel and Partners, Ltd, acted as Joint Engineers for the works. The design of the quay walls was inspired by, and was under the direction of, the late Dr K. W. Mautner, of Messrs L. G. Mouchel and Partners, Ltd.

Mr S. M. Dunsmore, A.M.I.Mech.E., was the Resident Engineer in the early stages of the work, and he was succeeded by Mr T. P. Kidman, B.Eng., M.I.C.E.

The contractors for the whole works were Messrs Holloway Brothers (London), Ltd, with Messrs K. L. Kalis, Sons & Co., Ltd, acting as subcontractors for the dredging.

The Author wishes to express his thanks to the Joint Engineers for their encouragement and permission to write this Paper, and to the individual engineers who have assisted with drawings and information concerning the design and construction. He also desires to record his appreciation of the skill and care displayed by the contractors, which ensured the satisfactory completion of these specialized works without injury or accident.

The Paper is accompanied by eleven sheets of diagrams and twelve photographs, from some of which the half-tone page plates, the folding Plates, and the Figures in the text have been prepared, and by the following Appendix.

APPENDIX

Types and Numbers of Caissons

Table 1.—original plan

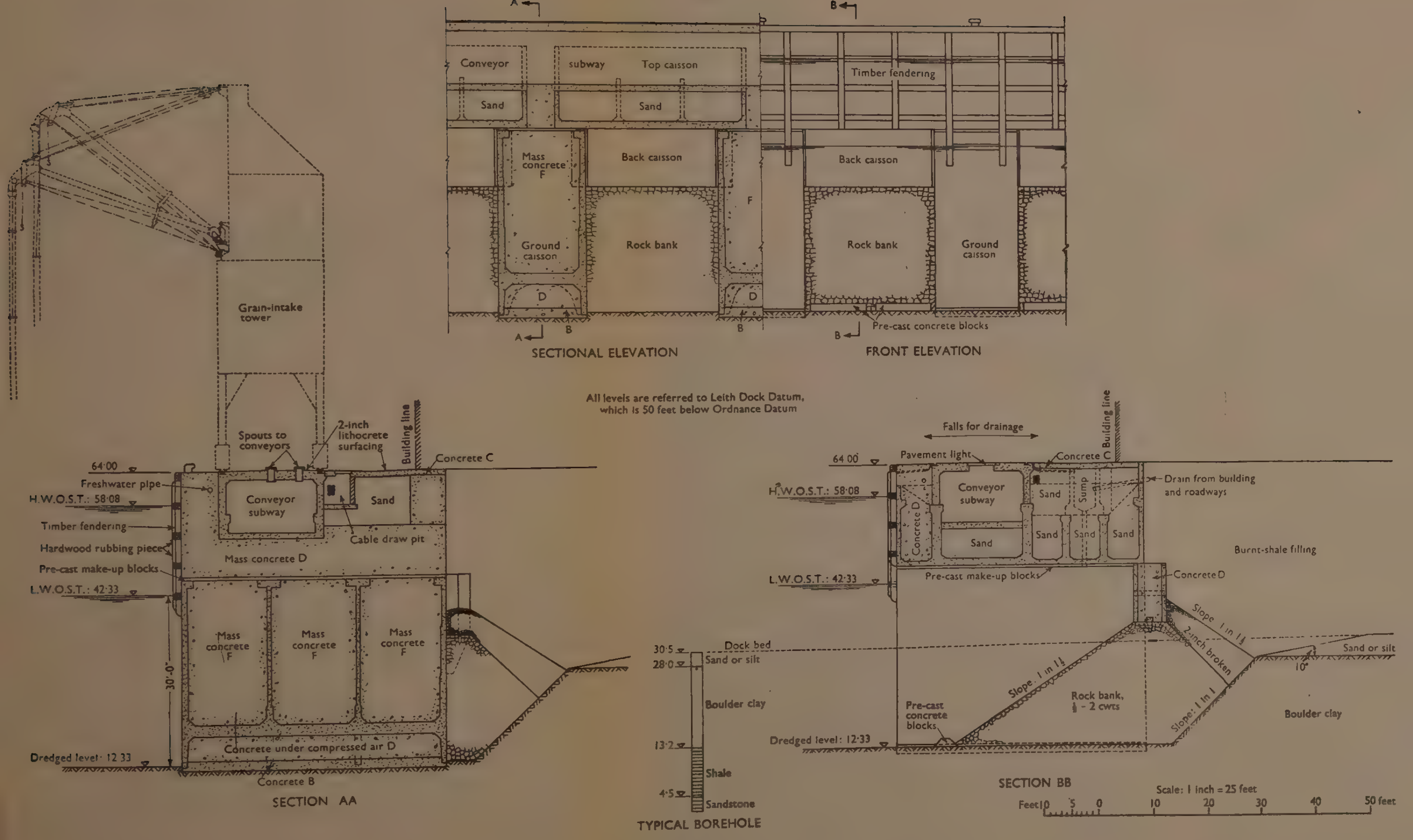
Position	Dimensions: feet		Number	Total
Ground	Large	45 × 15 × 33	14	
	Small	45 × 15 × 27·67	11	
	Corner	48·5 × 52·5 × 33	2	27
Back	Large	10·5 × 6 × 29	15	
	Small	7 × 6 × 24·25	11	26
Top	Large	45 × 33·5 × 14·67	15	
	Small	45 × 28·75 × 13	11	
	Corner	45 × 48 × 14·67	2	28
	Total number of caissons required			81

Table 2.—as constructed

Position	Dimensions: feet		Number	Total
Ground	Large	45 × 15 × 33	10	
	Small	45 × 15 × 27·67	11	
	Corner	48·5 × 52·5 × 50·17	1	
	Land	Various. Full wall height	3	25
Back	Large	10·5 × 6 × 28	11	
	Small	7 × 6 × 23·75	11	22
Top	Large	45 × 33·25 × 14·67	11	
	Small	45 × 29 × 13	11	22
	Total number of caissons required			69

THE DESIGN AND CONSTRUCTION OF THE NEW DEEP-WATER QUAYS AT LEITH

Figs 3



SECTIONAL ELEVATION

FRONT ELEVATION

All levels are referred to Leith Dock Datum, which is 50 feet below Ordnance Datum

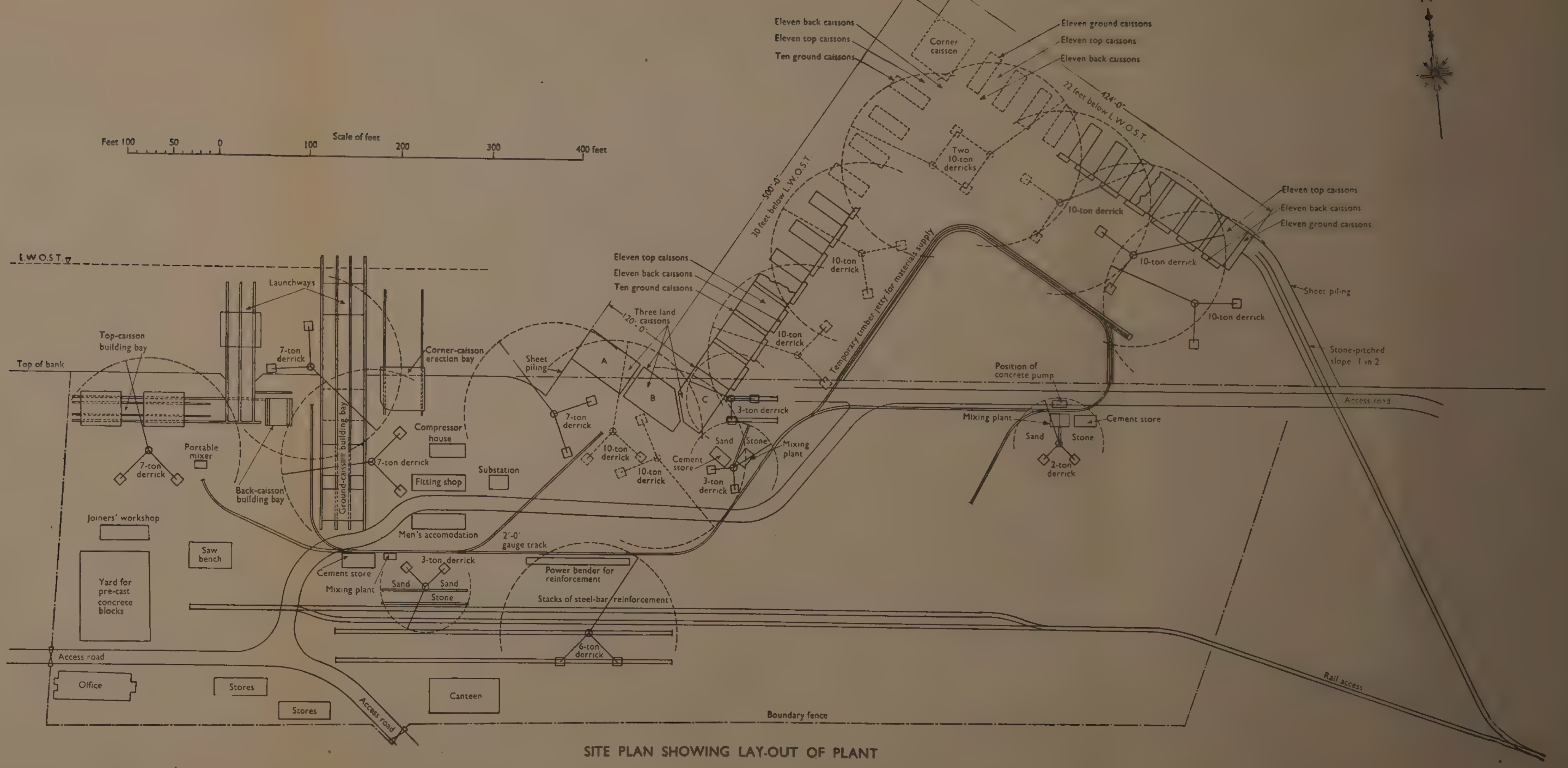
SECTION AA

TYPICAL BOREHOLE

SECTION BB

500-FOOT QUAY

Figs 4



SITE PLAN SHOWING LAY-OUT OF PLANT

The Institution of Civil Engineers. Proceedings, Part II, February 1952.

M. C. WHITE

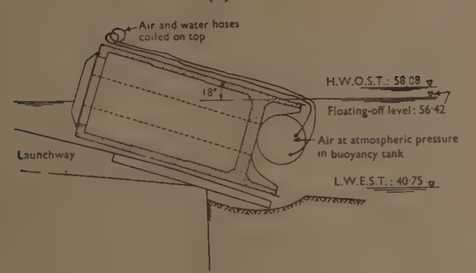
William Clowes & Sons, Limited: London.

PLATE 2
DEEP-WATER QUAYS AT LEITH

THE DESIGN AND CONSTRUCTION OF THE NEW DEEP-WATER QUAYS AT LEITH

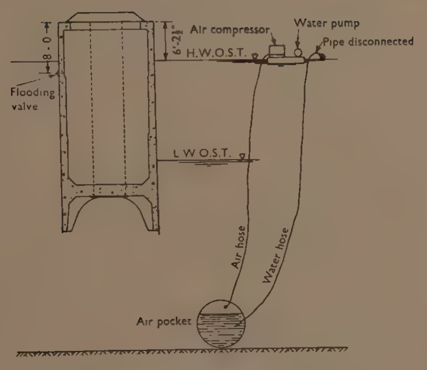
FIGS 8

(a)


Air and water hoses coiled on top
18°
H.W.O.S.T.: 58·08
Floating-off level: 56·42
Air at atmospheric pressure in buoyancy tank
Launchway
L.W.E.S.T.: 40·75

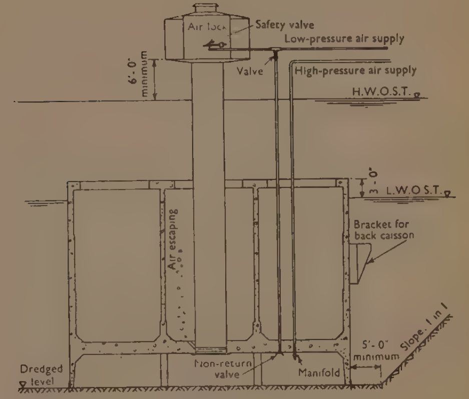
FLOATING OFF

(b)



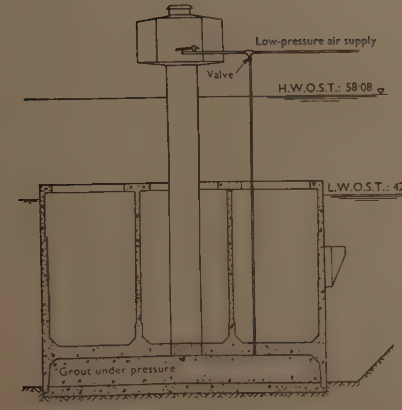
UPRIGHTED, RECOVERING BUOYANCY TANK

(c)



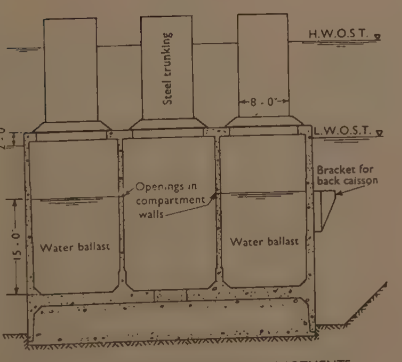
POSITIONING AT BERTH.
PREPARING TO DE-WATER WORKING CHAMBER

(d)



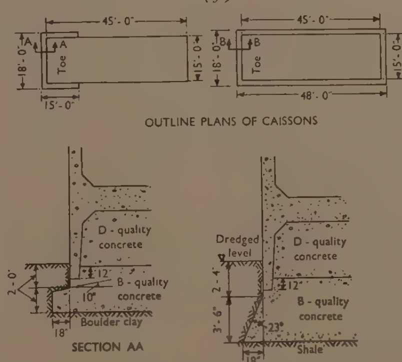
SINKING COMPLETED AND WORKING CHAMBER SEALED

(e)



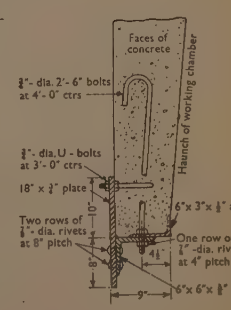
PREPARING TO CONCRETE UPPER COMPARTMENTS

(f)



UNDERCUTTING DETAILS

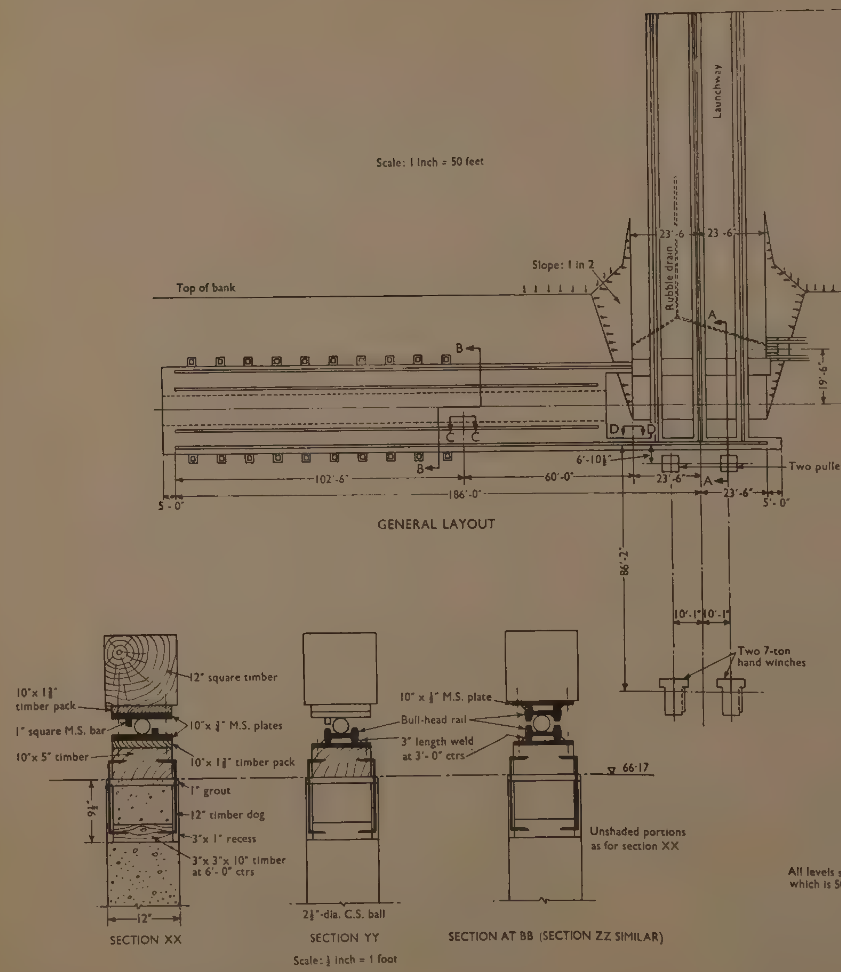
(g)

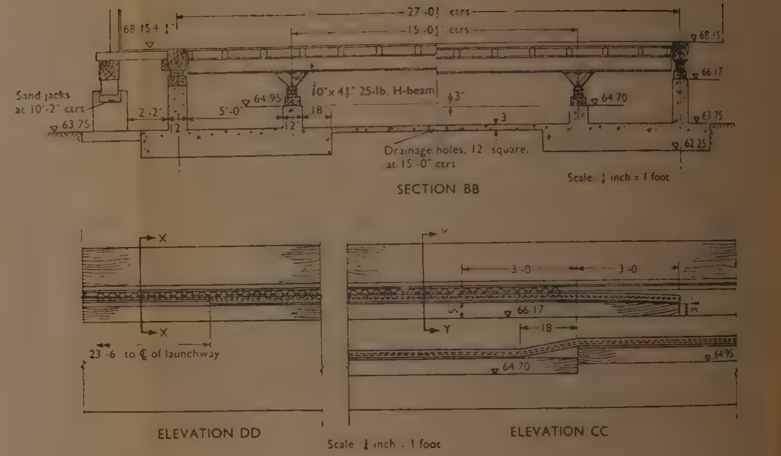

Faces of concrete
Haunch of working chamber
¾"-dia. 2'-6" bolts at 4'-0" ctrs
¾"-dia. U-bolts at 3'-0" ctrs
18" x ⅜" plate
10"
Two rows of ⅞"-dia. rivets at 8" pitch
6"x 3"x ⅜" angle
One row of ⅞"-dia. rivets at 4" pitch
4½"
8"
9"
6"x 6"x ⅜" angle

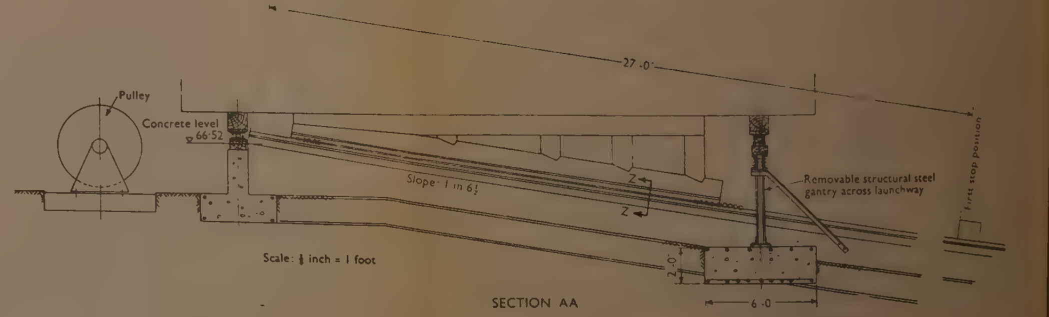
CUTTING EDGE

CONSTRUCTION OF GROUND CAISSONS

FIGS 11







All levels shown are referred to Leith Dock Datum, which is 50 feet below Ordnance Datum.

TOP CAISSONS – BUILDING BAY AND LAUNCHWAY

M. C. WHITE

The Institution of Civil Engineers. Proceedings, Part II, February 1952

WILLIAM CLOWES & SONS, LIMITED, LONDON

Discussion

Mr J. H. Jellett remarked upon the increasing tendency, illustrated in the Paper, to use concrete as a factory material for the production of components, which were then taken to the spot at which they were to be used. The practice of constructing quay walls and breakwaters by floating-out concrete caissons and sinking them either on or into the sea-bed was not new, although at the time that the Mulberry Harbour had been built many people had gained the impression that a new and somewhat revolutionary principle had been discovered.

At Leith, however, the top caissons had involved the pre-casting of almost an entire section of the quay, transporting it, and placing it on its foundations. That was an interesting development and, with the increasingly high cost of site and constructional labour, it would enable the dock engineer to take full advantage of all the latest developments in central mixing and batching—not always so readily applicable to dock and harbour construction.

Those members of the Division who had recently visited Le Havre would have noted the similarity between the application of that technique there, and in the works described in the Paper. The French engineers were doing something very similar in casting, on a specially prepared site, large sections of concrete quay which were then slid off into the water and taken to their appointed place, the substantial difference being that at Le Havre the pre-cast units were floated into position on a pontoon which was itself constructed of pre-stressed concrete and used over and over again. In the case of the works at Leith, each separate unit was made self-buoyant, but that was a difference only of detail, not of principle.

There were three points on which he would like to question the Author.

Had any observations been taken since the completion of the work on the level of the groundwater in the reclamation ? On p. 62 of the Paper the Author had stated that the horizontal forces were calculated assuming full hydrostatic head between high- and low-water levels ; and on p. 79 that the material used for the reclamation was good from the point of view of drainage. Was it not rather drastic to assume full hydrostatic head between the extreme levels of tide when there was reason to hope that the groundwater in the reclamation might follow the tide, not immediately but with not more than a certain lag between the two ?

What was the Author's final view of the value of the outside batter provided in the construction of the land caissons ? On p. 67 there were three references to this outside batter. It had been stated, first, that it required constant vigilance to maintain correct alignment ; secondly, that it obviated any need for kentledge ; and thirdly, that it proved troublesome by allowing " blows." Thus there appeared to be two disadvantages as against one advantage. If similar work had to be done, would the Author again use an outside batter ?

The third point concerned the subject of quay width. Mr Jellett assumed that the width of 40 feet was sufficient for the special purpose for which the quay had been designed. But at Southampton, where there was a quay width of 35 feet in the old docks and 50 feet in the new docks, visiting engineers from the Dominions and foreign countries had remarked, in surprise, on the narrowness of the quays. Further, those members who had visited Le Havre recently would probably have noticed the wide space between the edge of the quay and the nearest building to it in most of the new construction.

Mr W. Storey Wilson observed that under the heading " Acknowledgements," the Author had recorded his appreciation of " the skill and care displayed by the contractors," and in return, on behalf of the contractors, Mr Storey Wilson thanked the Author and the Joint Engineers, not only for approving the contractors' designs of the temporary works, but also for modifying the design of the permanent work, as necessary, to suit the methods of construction proposed by the contractors. That co-operation between engineers and contractors had contributed greatly to the successful completion of a difficult undertaking.

During the war, the contractors had been responsible for the construction and launching of a large number of concrete units of many kinds. At Stokes Bay, on the South Coast, they had built seven Phoenix units for Mulberry Harbour,[1] each weighing upwards of 2,000 tons. These had been built on sleeper walls and, from their building positions, had been moved forward to the launchway on trains of balls, and all seven had been launched successively within a period of 31 days.

When tenders had been invited for the deep water quays contract at Leith, the contractors, although not surprised, had been pleased to see that the design put forward for tender would give them the opportunity of again putting into practice the technique they had developed in the moving and launching of heavy concrete structures.

The Author had explained that because of the delay in delivery of special equipment, the contractors had suggested the replacement of fourteen floated caissons by three land caissons. That had not been the real reason; the real reason for the contractors' suggestion had been a common-sense one, for it had appeared fantastic to dredge a vast area of reclaimed land in which to float and sink fourteen caissons and, having done so, to backfill with imported material the area dredged, when in fact dry land was available on which to build and sink the caissons.

He had yet to be convinced of the necessity for pre-stressing the cutting edge of the caissons. The cables had been placed close to the steel cutting-edge; and he wondered what the steel cutting-edge had thought about being compressed in that unusual way! He had been responsible for sinking a large number of caissons under compressed air, of all sizes and in all sorts of conditions, and it would not have given him any misgivings had the pre-stressing been entirely omitted.

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[1] W. Storey Wilson and F. W. Sully, "The Traversing and Side-Launching of Heavy Craft." Civil Engineer in War, vol. 2, p. 211. Instn Civil Engrs, 1947.

The walls of the land caissons had been designed with a batter on the outside; that had been a great mistake and had considerably increased the contractors' difficulties during sinking. He supposed that in future people would come along with the wonderful idea of battering caisson walls to reduce skin friction, and it might be very difficult to persuade them against it; but he could give an assurance that battering the walls of a caisson to be sunk under compressed air was simply building up trouble for the person responsible for sinking it.

The Author had recorded as a feature of the design that, over the gaps between the ground caissons, the horizontal pressure from the filling was fully supported by the rock bank between the caissons. Mr Storey Wilson did not agree with that, and considered that the filling would wholly or partly arch over and bring the reactions on to the caissons.

The site plan (Figs 4, Plate 1) showing the lay-out of the contractors' plant indicated nine 10-ton derrick cranes, whereas there had been only two 10-ton cranes for the wall construction, and the lay-out showed the crane positions. The cranes were mounted each on three pontoons which, in turn, rested upon timber-piled stages, and the cranes were floated from stage to stage as the work proceeded. That had been an original idea proposed by Mr G. B. Gowring, the contractors' agent in charge of the contract at the time, and it had worked extremely well.

The Author had referred to the concreting of the caisson working chamber being carried out by concrete pump, with some degree of success. That method had been to some extent experimental at Leith, but had since been adopted elsewhere as standard practice; and on one particular caisson about 5,000 cubic yards of concrete had been pumped into the working chamber without any trouble.

He thought that the Author might have included in his acknowledgements the names of Mr. G. B. Gowring, Mr R. S. Read and Mr J. W. Reader, who at different times had been, each in turn, contractors' agent on the site responsible for the actual carrying out of the work; and Mr V. Silvera, who had done much of the detail design in the offices of Messrs L. G. Mouchel and Partners, Ltd, and who had been particularly responsible after the untimely death of Dr Mautner.

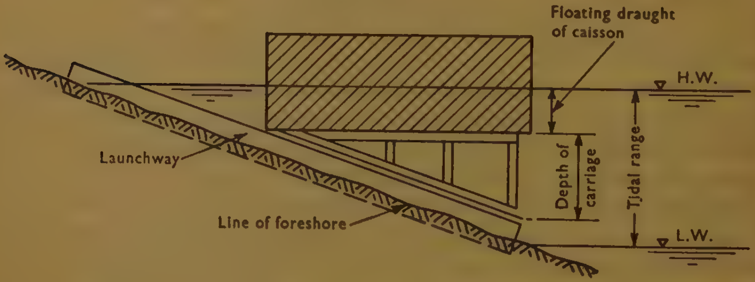
On behalf of the contractors, he wished to thank Mr Fortune and Mr Bunny of L. G. Mouchel and Partners; both of them had taken a large share in the principal deliberations, on both design and construction; and, if he had failed to convince them that shale was rock and should be paid for as such, he nevertheless very greatly appreciated the help they had given to, and the fairness of their treatment of, the contractors.

Mr G. B. Gowring said that, having been concerned with the launching arrangements described in the Paper on behalf of the contractors, he proposed to make a few remarks on the subject of temporary launchways in general and the problems which had had to be overcome at Leith, with the results which were described in the Paper.

Concrete caissons, unlike steel caissons, did not lend themselves to partial construction of the lower portion, which could be launched when the weight was small, followed by completion afloat so that the final weight did not have to be carried over the launchways. It had therefore been necessary, with one exception, to produce a scheme for complete construction on land and to devise launching methods which would handle the total weight of the caissons involved.

The exception had been the corner caisson, which was treble the size of any other ground caisson. If that caisson had been built entirely in reinforced concrete, it would have had a launching weight of more than 1,000 tons, which was 10 times the launching weight of the steel shoe as it had actually been built. It would also have governed the size of the launchway, as it would have been approximately 3 times the weight of anything else that had had to be launched.

Fig. 15



In preparing the launching scheme at Leith—in fact, in preparing any launching scheme—the rise and fall of the tide was the governing factor for both the temporary construction and the draught of the object to be floated. If the tidal range was insufficient, underwater construction became necessary, as on permanent slipways, and that involved cofferdams, divers' work and possibly dredging, which greatly increased the cost of the launchways; and therefore it paid to exercise all possible ingenuity to avoid it. At Leith there was a tidal range of approximately 16 feet at springs. Thus there was a depth of 16 feet in which to build the launchway and the carriage to carry the object to be floated; then the draught of the object itself had to be taken care of within the 16 feet rise and fall.

In the case of a unit which floated horizontally, the lowest position which could be reached before floating out was determined by the position of the inner edge of the bottom of the object in relation to the launchway (see *Fig. 15*). In the case of the ground caissons, it had been necessary to launch them on their side, because that had been the only condition in which there was sufficient depth of water near the shore to convey them to the dredged berth. When the ground caissons were floating vertically, they drew about 24 feet; when on their side, they drew between 15 and 16 feet. This was shown in Fig. 8 (*a*), Plate 2, from which it would be seen that the ground-level had been such that there was only just room for the cutting edge to float-off without striking the ground in front of the launchways. In fact, a small portion of the ground had had to be dredged away in order to accommodate the bottom of the launchways. That caisson could not possibly have been launched in a vertical position and had had to be launched in an inclined position in order to get it floated into its position.

The natural angle of flotation of the Leith ground caissons in the original contract design had been approximately 18 degrees, and that had produced a convenient lay-out for the lower end of the launchway. In those days the caisson had had a 1-foot roof to the working chamber. If the caisson had been floated-off and taken into the dredged berth in that condition, it would have been necessary to have added some form of ballast to it in order to get it upright, and it had originally been intended to flood the upper compartments with water to induce it to float in a vertical position. However, that had been considered to be undesirable because the water ballast was rather unstable, and whether it would have uprighted itself successfully had been a little open to doubt. Dr Mautner had therefore been reluctant to pursue that scheme further.

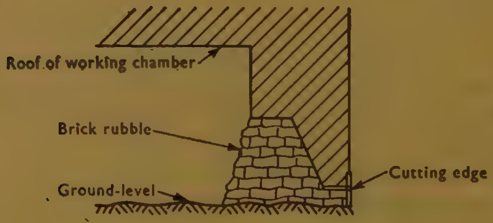
It had then been decided that if the roof of the working chamber were thickened up, the caisson would naturally float in a vertical position, and that in fact had been done.

The launching position in that angle having been agreed, consideration had then been given to the building position. It had been decided that it would be very awkward to build the caisson at an inclination, since the vertical walls would have been at that inclination, and all the shuttering for those inclined walls would have been very much heavier than the shuttering for vertical walls; furthermore, it would not have been possible to true-up the shuttering by means of levels and plumb bobs without complicated batter rules. The conclusion had been reached that the only practical way was to build the caisson horizontally on its side. Thus there had been the problem of tilting the caisson from its building position to the inclined position for floating off.

Another factor had been that, for the programme, it had been necessary to launch a caisson roughly every 2 weeks, and it had taken about 6 weeks to build a caisson. It had therefore been necessary to have either three building berths with their separate launchways, or else some method by which the caissons could be built in succession and moved to the launchway. The solution adopted had met both difficulties: it had enabled the caisson to be built horizontally and it had enabled it to be tilted for floating off. The use of hard steel balls rolling on steel rails had been practised for many years for rolling in bridges where quick replacement was necessary, as, for example, on railway work. These were always used on the level or nearly on the level; and to enable them to be used on steep gradients the balls were retained by perforated plates connected to each other to form a continuous band. The plates kept the balls at the correct spacing, they prevented the balls from running down the slope, and they also fitted the balls in under the carriages, even if the carriages were submerged.

The launching method had worked without mishap, and in fact had become so easy that one caisson had launched itself! On a rough day, it had been decided to postpone the floating-off, and the caisson had been left on the launchway about 2 feet above the floating-off level. That night the tide had been exceptionally high and the next morning the caisson had been found on a mudbank about ¼ mile away from the launchway. Luckily there had been a following high tide, and tugs had brought the caisson back into its position—and there had been no further mishap.

Fig. 16



The sinking of land caissons was made rather difficult because of the fact that, if a reinforced-concrete caisson had to be sunk it was necessary to build up such a height before sinking was started that the structure might become too heavy for the ground on which it was built. The first land caisson had sunk 4 feet at one end when the roof shuttering had been stripped. Better control had been established on the others by introducing a step in the haunching and packing some brick rubble under it (*Fig. 16*). The rubble had been removed after the caisson had sunk through the soft ground and into the hard material. That expedient had overcome the difficulty which had been experienced with the first land caisson.

Mr P. R. Robinson agreed with Mr Jellett that the Paper brought out particularly the increasing importance of the method of using pre-cast units and bringing them on to the site, although it did not necessarily follow that this method was cheap. In order to obtain full advantage of it there must be a certain amount of repetition, in addition to mere prefabrication.

He said that attention had already been drawn to the width of the quay being 40 feet. That did appear to be somewhat narrow, but was probably related to the fact, as stated on p. 8, that the design of the caissons had been governed by the assumed requirement of using the backs of the caissons for carrying the foundations of future buildings. To incorporate that requirement into caissons of 50 or 60 feet width would have been uneconomic.

He remarked that the cost of the quay—£450 per foot—seemed to be very high. Furthermore, if the deciding factor in the adoption of this caisson type of quay had been to get the foundations of the building on the caissons then, taking as much as £350 per foot as the average cost of a false type of quay, it would appear that the foundations of two walls of the building had cost at least £100 per foot—which also seemed to be very high. In his opinion, it would have been cheaper to construct the buildings entirely separate from and behind the quay.

He presumed that the method of constructing caissons in lagoons had been considered. There appeared to have been a fair amount of ground available, and the method of launching caissons, good though it was, particularly when there was restricted ground, was not necessarily cheap.

He thought that a dipper-type dredger as used extensively in America for heavy dredging, might have been more suitable to deal with the hard material (boulder clay, shale, and sandstone) encountered, although the bucket-type dredger was in common use in Great Britain. Incidentally, the dredging contractors had developed a type of rock-chisel which they had used with considerable success in somewhat similar soft rock, but he believed that at the time of the work at Leith the machine had been busy in France.

Mr D. H. Little said that for the past 10 years he had been associated with the design of something like fifty jetties and wharfs. Not all of them had been built and not all of them had been big ones, but all of them had been design problems, and the solution in every case had been a relatively simple and crude sort of pile structure. They had not been so interesting, as the design which the Author had described—a very clever and somewhat intricate design.

When Mr Little read about such interesting jobs, his first reaction was one of dissatisfaction with himself, for he felt that he was getting into a rut; but the organization for which he worked could not afford the sort of things that had been described! So he had to content himself with the thought that clever designs might be good things from the designer's point of view but that unless they were really economical, they could not be called clever engineering.

At the end of the Paper the Author had given the costs and Mr Little would like to especially thank him for this. The cost was £450 per linear foot, and since the overall width of the jetty was 45 feet, it worked out at £10 per square foot. A simple piled structure would have cost about £5 per square foot. So far as he could gather, the big sea terminal at Fawley had cost £1,600,000, and the area of the jetty was 260,000 square feet, so that the cost worked out at a little more than £6 per square foot. Bearing in mind the very long approach, he would have said that a plain piled structure would have cost something less than £5 per square foot.

Before the war there had been a Paper[1] on Southampton Docks, where monoliths had been sunk, and he did not think the Author of that Paper had been very satisfied that the result was economic. Mr Little knew of slightly bigger caissons that had been built at about the same time, and those had not been too cheap. He thought it was also significant that continental designers were all turning away from gravity walls.

He thought that the long-term value of the Paper, apart from any intricate and interesting factors described in it, was that it was just another warning of what not to do unless one jolly well had to do it!

To maritime engineers, he would say "If you cannot be good, be simple."

Mr W. H. Maggs asked whether the Author could give some details of the performances and size of the dredgers and hopper barges.

Mr John Palmer asked how it was proposed to use the short length of quay end (which was shown as a chain-dotted line) jutting out between the newly-built quay walls and the quay marked "future quay walls" in *Fig. 1.*

Mr I. S. Chisholm who had been employed at Leith before the war in connexion with the construction of the breakwaters, referred to the Author's statement that the maximum wave-height was only 2 feet, which seemed to be a tribute to the designers of the breakwaters. The fact that the wave-height was relatively small would be invaluable in developing the port, and although in certain cases it was necessary to have ships loading and unloading where the water-level did not vary very much, the variation of the tidal range should not be serious in trades of the type described and the cost of constructing and maintaining an entrance lock and gates had been obviated.

He would be interested to hear from the Author what delays had been caused by weather conditions, even with that small wave-height, particularly in placing the upper caissons. The movement of the derricks on pontoons with very little trouble emphasized that the conditions appeared to be ideal for the method of construction adopted at Leith.

Mr Harry Ridehalgh said that, although he agreed with Mr Little regarding the cost of £450 per foot, he thought that advice to engineers to be simple should be accompanied by constructive suggestions.

The Author had stated in the Paper that piling had been impossible. Figs 4, Plate 1, showed steel sheet-piling in certain places, and although these were described as temporary, the fact remained that piling had been driven. Could an assurance be given that king piles could not have been driven with any degree of certainty?

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[1] M. G. J. McHaffie, "Southampton Docks Extension." J. Instn Civ. Engrs, vol. 9, p. 185 (June 1938).

The borehole shown in Figs 3, Plate 1, appeared to indicate that the boulder clay did not go down to ultimate dredged level. The shale could be removed by dredger, and therefore it was not unreasonable to assume that the shale could have been penetrated by piling. Would it not have been feasible to dredge off the boulder clay to uncover the shale? Piles could then have been driven into the shale to support a piled structure of the £5-per-square-foot type to which Mr Little had referred.

Alternatively, he suggested, a coffer dam could have been built, enabling the quay to be constructed in the dry, and resulting in a much lower cost.

Mr Ridehalgh emphasized that he was making tentative suggestions only, which had no doubt been considered, but would the Author please state why they had been discarded?

⁂ **Mr R. W. Bishop** observed that the River Forth and its tributaries drain about 900 square miles of land, and the greater part of the suspended matter in the water seemed to be deposited in the tidal reach between Grangemouth and the Forth Bridge. Investigations had revealed that, in the Leith Roads, about 20 miles down the estuary, the silt charge of the water was about 16 parts per million by weight on spring tides and 6 parts per million on neap tides; the average size of the particles was 16μ, which, according to the M.I.T. (Mass. Inst. Tech.) classification,[1] was a medium silt.

The new quays described in the Paper had been built in an artificial tidal basin sited on the south bank of the bell-mouth entrance to the Forth and, in other circumstances, the triangular shape of the basin, with its comparatively narrow entrance at the apex, would be conducive to excessive silting; to maintain a navigable channel with a depth of 18 feet of water at low tide up to and alongside the new berths, would involve considerable dredging. However, at Leith one would imagine that owing to the negligible silt charge of the Forth, that was not so; but the Water of Leith might be the deciding factor, especially as its regime might be altered by the new works proposed, and it would be interesting to have the Author's remarks on that point.

Mr J. H. Brooks observed that the Author had made only passing mention of the fundamental decision—for better or for worse—which would govern the future development of the Port of Leith; namely, the choice of a tidal, as distinct from a locked system for the future growth of the port.

Whilst the former had been adopted by many of the great ports of the Continent, the decision in the case of Leith, with its 16-foot range at spring tides, must have called for a careful weighing of the pros and cons.

No doubt, in the case of a port such as Leith where expansion was on reclaimed land, and growth seawards was a possibility, the tidal system

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⁂ This and the following contribution were submitted in writing.—SEC. I.C.E.

[1] R. Glossop and A. W. Skempton, "Particle-size in Silts and Sands." J. Instn Civ. Engrs, vol. 25, p. 81 (Dec. 1945).

offered special attractions, and allowed an economical, piecemeal development of quayage within an area adequately protected by enveloping breakwaters.

Modern developments in materials and design, such as the pre-stressed-concrete pile, heavier-section steel sheet-piling, and designs of the type employed at Leith, permitting of greater depths of quay wall without excessive increase in cost, might well mean that the use of a non-tidal system, with the attendant disadvantages mentioned by the Author, would recede in popularity.

It might be that the Author felt that consideration of that matter was too lengthy for inclusion in the Paper, but Mr Brooks believed that a résumé, even if brief, of the factors influencing that bold decision, would be of great value to those concerned with port development.

The Author, in reply, referred first to Mr Jellett's remarks regarding the modern tendency to make increasing use of pre-cast units. That practice, with its more exact control of the concrete mix at a central batching plant, had much to recommend it, particularly on tidal work. By constructing the top caissons ashore, the shell of the main quay-wall superstructure had been completed to above high-water-level and that had enabled the balance of the " in-situ " concreting to be completed in the dry without much difficulty.

There was a strong resemblance between the interesting quay-wall constructions at Le Havre and those at Leith but, in addition to the one difference in detail mentioned by Mr Jellett, the Author would refer to the more cautious approach by the British engineers in the use of pre-stressing. In 1945 and 1946 it was a new technique when applied to marine work and, at Leith, it had been adopted as an expedient to obtain additional strength for a special purpose and for a limited period of time, whereas at Le Havre, the pre-stressed members formed an integral part of the permanent structure.

With regard to the design, Mr Jellett had asked if it had not been rather drastic to assume full hydrostatic head in the calculations, and had inquired if any observations of groundwater-levels in the reclamation had since been made. The Author was of the opinion that that assumption gave a comfortable margin of safety, and pits, sunk for the foundations of the new mill buildings, showed that at present there was complete drainage through the burnt-shale filling down to at least half-tide-level. There was, however, a slight lag in the drainage and, in time, it was possible that the " fines " in the shale might form a blanket to retard still further the speed of drainage and, in the short time available between tides, that might produce the condition visualized by the designer.

Mr Storey Wilson had expressed his dislike of the outside batter to the walls of the land caissons. The Author agreed that 12 inches in 11 feet was excessive, and had proved very troublesome to the contractor. On the other hand, when sinking to a considerable depth through tough materials, such as the boulder clay at Leith, he was inclined to the view that a slight batter, not exceeding 2 inches in 10 feet, would prove beneficial.

Mr Jellett and Mr Robinson had both commented on the 40-foot apron or quay width. This was the width which had been specified, and was adequate for the special purposes for which the two quays were designed; namely, the discharging of bulk grain cargoes and the loading of bagged or bulk products from the mill and warehouse. At one quay the equipment would comprise travelling pneumatic intake-towers with a subway for the conveyor bands, and at the other the loading would be accomplished by overhead conveyors. The apron was intended to carry considerable road traffic but quayside cranes were not required, and the railway sidings were confined to the opposite side of the mill buildings.

There was a great variety of opinions on the subject of quay widths but, in fact, a suitable width was generally regulated by the nature of the traffic, the method provided for the loading or discharge, and the type of transport required for collection or distribution of the goods. During the recent modernization of one quay at Leith a width of 36 feet had proved to be adequate for two lines of railway sidings, with portal cranes spanning the quayside track.

The Author was pleased that Mr Storey Wilson had referred to the spirit of co-operation which had existed between the engineers and the contractors; this had, in large measure, contributed to the successful completion of the works. With regard to the omissions from the acknowledgements, the Author agreed that, owing to the extent and value of their work, the names of the four gentlemen mentioned by Mr Storey Wilson might well have been included. Many persons had been associated with the design and construction, and the Author had confined his acknowledgements, perhaps unwisely, but after due consideration, to the names of firms and the principal Joint Engineers. He wished to thank Mr Storey Wilson for the frank expression of his opinions regarding certain aspects of the design, and particularly for his reference to the derricks which were floated from stage to stage for the wall construction.

The Author had listened with interest to Mr Gowring's remarks concerning caissons and launchways in general, and to his description of the special problems he had encountered and overcome at Leith by the ingenious methods devised for launching the caisson units. It only remained for him to thank Mr Gowring for amplifying all the points in the Paper concerning launchways and launching.

Mr Robinson, Mr Little, and Mr Ridehalgh had all referred to the cost of the quay walls which, including dredging for the walls, had worked out at £450 per linear foot; other costs had been quoted for comparison. The Author had expected this figure to provoke interest and discussion and he would agree that £300 to £350 per linear foot was a fair estimate for an average quay wall with a height of about 52 feet. Average costs could be most misleading, especially when applied to marine structures, and he was convinced that, on that particular site, any other type of quay wall would have been more expensive to construct and would probably not have fulfilled the special functions required. It was possible that using the experience gained, certain modifications might be made in future constructions to reduce the cost.

As stated in the Paper, any normal form of piling was precluded by the nature and varying levels of the underlying boulder clay with outcrops of shale and sandstone, but attention had been drawn to the short lengths of steel sheet-piling which had been driven at the ends of the quay walls. This seemed to conflict with the previous statement, but the explanation was simple: overlying sand was not dredged away at the temporary connexions to the land and, although penetration into the boulder clay averaged only about 3 feet, it was possible in those positions to provide anchorages with supporting tie-rods. It was quite safe to state that king piles could not have been driven into the outcrops of hard sandstone and shale which mostly occurred at, or about, dredged level. Even assuming that piles of some description could have been driven into the softer strata, the problems of bending moment and anchorage, especially at the external corner, would have proved both complicated and costly. Monoliths could not be sunk in this type of ground and a cofferdam construction, although it had been considered, would have been practicable only at the corner occupied by the land caissons.

Mr Robinson had suggested that a dipper-type dredger might have been more suitable to deal with the harder materials. This was probable, but the Author had had no experience of this plant and was not sure what might happen if the bucket were to strike a very large boulder.

Mr Palmer had asked about the probable use of the future 500-foot cross-berth shown chain-dotted at the end of the dredged channel in *Fig. 1*. Although the outline of the future quay walls was subject to alteration it was the intention to make this a fitting-out berth equipped with a large crane.

The Author agreed with Mr Chisholm that the new breakwaters had proved most effective in reducing wave-action and their design was a tribute to the careful model-experiments carried out at Delft Hydraulic Laboratory. Delays due to weather conditions had been small and were caused more by wind than wave-action.

Mr Maggs had requested some particulars regarding the output performance and plant employed by the dredging sub-contractor. Since there had been a limited area to be dredged, only one dredging unit could be operated satisfactorily at the one time but this had been adequate to deal with the relatively small total volume of materials which had to be removed. This unit was a robust bucket-dredger, of the stationary-type with cast-steel buckets of 28-cubic-feet capacity, belt-driven by a 400-indicated-horse-power triple-expansion steam-engine. The spoil was discharged into dumb hopper-barges, each of 500-cubic-yards capacity, with steep well sides and equipped with hydraulic lifting gear for the bottom doors.

Reliable output figures cannot be given owing to the wide variation in results obtained when working in the different strata, but, including breakdown delays, the total dredging time to remove the 250,000 cubic yards of mixed materials was 52 weeks. Despite the wide experience of the dredger crew, the ladder chain had been broken on five occasions and many buckets severely damaged or broken in the course of the operations.

Mr Bishop had suggested two sources of probable silting in the artificial bay formed by the new breakwaters, but the Author was not unduly perturbed by the amount of silting which had taken place since their construction.

By reference to *Fig. 1*, it could be seen that deposition of matter carried in suspension by the Water of Leith might be expected to occur where the widening of the channel, opposite the existing dock and dry-dock entrances, caused a reduction in the velocity of the stream. That, in fact, did take place, and the regime of the small stream had not been substantially altered, although it was anticipated that the shortening of the west pier would have a slightly adverse effect.

Siltage from the suspended material in the Firth of Forth was negligible, but a more serious item from that source might have been the littoral drift of the sea-bed, which on that shore moves from east to west. That problem was carefully investigated when the breakwaters were designed and was successfully overcome by the relative positions selected for the breakwaters and breakwater heads.

A small bank of sand tended to build up near the entrance behind the east breakwater, and had to be removed from time to time. The success of the breakwaters in performing their functions and the comparative insignificance of the silting problems may be gauged from the fact that, over the last 7 years, all dredging maintenance at Leith had been undertaken by one grab dredger.

The Author agreed with Mr Brooks that the decision to construct tidal quays in the new area was of great importance, but the subject was considered to be scarcely relevant and too involved for inclusion in the Paper. Some of the principal reasons which influenced the decision were described by Mr Brooks, others were mentioned in the Paper, and they might be briefly summarized under eight headings :—

1. Economical piecemeal development of the area without a large initial unproductive capital expenditure for a lock entrance, to which must be added the heavy operating and maintenance charges.

2. The piecemeal development, so far as could be seen, would be spread over many years.

3. Freedom of the area from silting and wave-action.

4. The short and easy approach channel from deep water into the area.

5. Free access for shipping could be obtained at all states of the tide by further dredging.

6. The vulnerability of large lock entrances to damage by shipping and also possible damage in the event of hostilities.

7. The experience of stevedoring firms and shipping companies at tidal berths in the outer harbour had been quite satisfactory.

8. The improved engineering technique for the construction of deep-water quays.

The decision had been taken before the Author had had any connexion with the port and, although plans existed showing various arrangements of proposed lock entrances to the area, he was of the opinion that the final decision was correct. In fact, unless there was a requirement for a sudden and extensive development of the whole area, he considered that the capital cost of a lock entrance could not be justified.

RAILWAY ENGINEERING DIVISION MEETING

13 November, 1951

John Cumberland Landale Train, M.C., M.I.C.E.,
Chairman of the Division, in the Chair

A discussion on the subject of

"Economy in Railway Civil Engineering"

was initiated by four Introductory Notes and, upon the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Introductory Notes:

(1) "**Economy through Labour Productivity and Incentive Schemes.**" By G. C. Stevens.
(2) "**Economy through Organization.**" By J. Taylor Thompson, M.C., M.I.C.E.
(3) "**Economy by Supervision.**" By A. K. Terris, B.Sc., A.M.I.C.E.
(4) "**Economy through Design.**" By P. S. A. Berridge, M.B.E., M.I.C.E.

Introductory Note No. 1

"Economy through Labour Productivity and Incentive Schemes"

by

George Charles Stevens

The present-day need to apply technical skill in the economical utilization of manpower, quite apart from the aspect of mechanization, cannot be overlooked by the railway civil engineer.

With full employment throughout Great Britain, and the trend of labour to prefer modern factory conditions to the all-weather heavy manual work which must be associated with railway civil engineering, it is important that the labour available to industry is used economically and to the very best advantage.

This Note deals almost exclusively with economies, showing what can be, and indeed has been, achieved by the payment of money additional to standard wages, for work carried out in excess of a standard task. Many will insist that there is something immoral about this principle, but until such time as all men are equal, both physically and mentally, methods of payment-by-results will continue to produce results whilst arguments about morals are likely to be inconclusive.

It must be clearly understood that whilst incentive schemes can achieve excellent results in economy of labour, high productivity, and as a means of retaining labour, they can also be very much a "two-edged weapon." Once an incentive scheme has been introduced its withdrawal is almost impossible, and any modification to an existing scheme is most difficult to negotiate. Considerable sums of money are involved; for example, a group of 100 men, may, in 12 months, have their total earnings increased by as much as £8,750 by incentive payments. It is therefore very important that engineers with some training in the basic principles of the study of human effort, namely, time study, should be consulted before embarking on any scheme.

In the past many incentive schemes have been introduced which were based on the existing productivity of the group of men concerned, and extra payments have been made over and above such productivity. At first glance this broad principle might appear reasonably sound. Assume, however, that there are two groups of men ("A" and "B") on similar work, with Group "A" better organized and working harder than Group "B"; such a scheme is unlikely to be popular with Group "A" unless a method is found of introducing a compensating factor.

The foregoing example points to the need of devising a standard measurement of work done, and it is this standard measurement or allowable time per operation, and the method of arriving at it, which is the very foundation upon which all good incentive schemes should be based.

The allowable time, generally referred to as the "Allowable Minutes" for an operation (the A.M. value), must make provision not only for the actual carrying out of the operation but also for the personal needs of the operator plus a factor as cover for relaxation to compensate for undue fatigue.

Just how to establish a standard time, or A.M. value, for an operation is the next problem, especially since several studies will have been made, all of which may be different.

The solution is to break down the time study into small units of, say, 60 seconds and effort-rate each time-unit of work. If we assume that maximum effort is graded at 100 and minimum effort, that is to say, the slowest speed an operator can work without going deliberately slow, at 50 then each small element of work can be rated, and if the results are normalized to a common basis of 60 we have a means of establishing a standard.

Table 1 illustrates the method; the lower number is the actual time taken and the higher number is the rating factor.

Whilst three different studies by three different engineers on three different operators would hardly give the exact results illustrated above, the summation of a number of studies should give results within 5 per cent.

TABLE 1

	Operation	Actual time :		Normalized time :	Units of 60 secs	
		mins	secs	secs	mins	secs
Study 1	A 60^{90} 60^{90}	2	00	180	3	00
	B 60^{80} 60^{75} 60^{90} 30^{90}	3	30	290	4	50
	C 60^{100} 60^{95} 45^{90}	2	45	262	4	22
		8	15		12	12
Study 2	A 60^{60} 60^{60} 60^{60}	3	00	180	3	00
	B 60^{55} 60^{60} 60^{75} 60^{60} 40^{60}	4	40	290	4	50
	C 60^{70} 60^{70} 60^{70} 52^{60}	3	52	262	4	22
		11	32		12	12
Study 3	A 60^{80} 60^{80} 12^{100}	2	12	180	3	00
	B 60^{80} 60^{75} 60^{85} 50^{60}	3	50	290	4	50
	C 60^{80} 60^{80} 60^{80} 22^{60}	3	22	262	4	22
		9	24		12	12

Once A.M. values have been established covering the work of a group of operators it becomes an easy matter to establish a measure of productivity per hour, merely by totalling the amount of measured work performed per day and dividing by the total actual hours worked. Appendix I is a typical example of an operator's Daily Work Sheet showing the calculations involved in assessing the individual's bonus earnings and the productivity factor.

Research and experience have shown that men will not give of their best for a financial incentive equal to less than one-third of their normal wages and that the average " best " of a group of men is approximately 80 A.M.'s of work per hour. The next step, therefore, is to find a convenient method of payment whereby men earning a weekly wage of, say, 120*s.* and working at a rate of 80 A.M.'s per hour are paid a bonus approximating to 40*s.* per week.

The basis of incentive payment described leaves the establishment of the level above which additional payment shall be made quite open. This is a very necessary feature since it is difficult to persuade a man when first introduced to a system of payment-by-results to work harder unless some immediate reward is forthcoming for his extra effort. A common productivity factor for straight day-work varies between 35 and 45 A.M.'s per hour. Assuming, therefore, that the basic task is set at 60 A.M.'s per hour and that the individual is working at a steady 40 A.M.'s per hour, then it will be seen that a 50-per-cent increase in productivity is required before any bonus payment can be earned. It is important, therefore, that careful consideration should be given to the existing productivity factor of a Works or group of men, before establishing a basic task, in order to ensure that small increases in productivity receive some immediate reward. Table 2 shows how incentive rates can be varied when applied to basic tasks.

TABLE 2.—BONUS PAYMENTS PER 10 A.M.'S PER HOUR

Performance of working	Basic from a 40	Basic from a 50	Basic from a 60
50	2¾*d.*	—	—
60	5½*d.*	3⅝*d.*	—
70	8¼*d.*	7¼*d.*	5½*d.*
80	11*d.*	11*d.*	11*d.*

It will be noticed that 11*d.* per hour at an 80 level amounts to 40*s.* 4*d.* per 44-hour week and that the figure per 10 A.M.'s can be varied to comply with the one-third-of-salary rule.

It can be argued that it is wise to fix the basic starting level at 60 A.M.'s per hour and to introduce various starting allowances to compensate for low initial productivity during the early stages of the scheme. Long experience of incentive payments applied to railway civil engineering may well reveal that a compromise scheme is the better proposition; that is to say, an allowance varying between 5 and 10 minutes may be conceded for a specific period of 4 weeks—10 minutes for the first 2 weeks, 5 minutes only for the third and fourth weeks, and thereafter the allowances would be eliminated altogether. Table 3 is based upon the assumption that productivity was at a level of 30 A.M.'s per hour prior to the introduction of an incentive scheme, and shows the starting allowances to be added for varying hourly performances, to give immediate financial reward for increased effort.

TABLE 3

Performance : A.M.'s/hour	Temporary starting allowance required : mins
40	10
50	20
60	30

The method of incentive payments previously described can be expected to increase productivity very considerably and without large capital expenditure. Increases in productivity of over 100 per cent are not unusual whilst increases of between 50 per cent and 60 per cent are the general rule. These increases do not reflect in any way on the efficiency of local or higher management prior to the introduction of incentive schemes, but are typical of the results obtained in some commercial concerns. The real cause of low productivity on straight day-work is a vexed question and certainly not within the scope of this Note.

Lest it should seem that too rosy a picture has been painted of the application of incentive schemes, and also to amplify the earlier remark that they can be a "two-edged weapon," it is only fair that attention should be drawn to the following points :—

(1) The question of bonus payments to supervisors has yet to be solved, and there are many cases of operators earning up to £1 per week more than their supervisor. This position is worsened by the fact that incentive schemes invariably lead to the curtailment of overtime working with a consequent drop in earnings of supervisors.

(2) Assuming that a satisfactory solution is found to problem (1), will not a similar position arise affecting those members of the managerial, technical, and clerical staff intimately concerned with the administration of incentive workers?

(3) It is necessary to ensure that increased production can be absorbed. The fact has to be faced that over-production or staff redundancy at one works can adversely affect the introduction of incentive schemes at neighbouring works.

(4) Consideration must be given to the employment of additional administrative staff so as to ensure that an incentive scheme, good at its inception, continues to remain a good one. Simultaneously, it must be borne in mind that machine improvements, minor re-organizations, new methods, the introduction of jigs, etc., entail revised A.M. values and not just increased bonus earnings for the operators.

Cases are actually known to the Author of incentive schemes which were basically sound and good when first introduced, but because they have been allowed to remain uncontrolled over a period of 6 to 7 years have resulted in high bonus earnings being paid for low productivity.

It will be appreciated that once an incentive scheme has been introduced the A.M. value of all standard work is known. Consequently it is possible to allocate with great accuracy sufficient numbers of operators to complete specific jobs. The average productivity of an individual or or a group of operators quickly becomes known, and the clock-hours necessary to complete tasks can be arrived at by dividing the task A.M.'s by the average productivity, that is to say,

$$\frac{\text{A.M. value per operation} \times \text{No. required}}{\text{average productivity}} = \begin{array}{r}\text{Clock hours necessary to} \\ \text{complete ``task.''}\end{array}$$

Each operator working under an incentive scheme is required to complete a Daily Work Sheet, and upon this sheet is inserted not only a record of tasks accomplished but of delays occasioned by wet weather, machine breakdowns, power cuts, waiting materials, or instructions, etc. This information is extracted from each Daily Work Sheet; the particulars are summarized and details inserted on Daily and, subsequently, Weekly Analysis Sheets which are prepared for each section of the Works (see Appendices II and III). Also included on each Weekly Analysis Sheet are particulars of the total wages paid to the men employed within that section, covering the hours actually worked. It is therefore possible for the management to exercise a day-to-day and week-to-week control of each section within a department.

Up-to-date information is essential to modern methods of works control to ensure that the causes of falling productivity are known immediately they occur and to enable management to take remedial action. This is in striking and refreshing contrast to the old methods, familiar to most of us, whereby the efficiency of a works was measured by figures produced by an accountant—often several weeks after the usual monthly period had elapsed.

The standard cost for each principal type of operation covered by an analysis sheet is measured in terms of pence per 1,000 A.M.'s produced. This is based on the A.M.'s produced at the target rate of 80 A.M.'s per hour, and the wages cost at day-work rates plus bonus appropriate to the target performance. The weekly analysis of labour utilization built up from the daily work sheets shows the output and wages cost per 1,000 A.M.'s produced. The weekly comparison of actual costs with the standard enables a close watch to be kept on production costs in each section of the department. To this standard cost can be added the cost of materials, according to the specification, to give a standard cost in labour and materials for each unit.

Overhead expenses covering clerical and management costs, maintenance and depreciation of plant and equipment, national insurance contributions, holidays with pay and allowances for unproductive time due to major stoppages occasioned by weather can be assessed for the period of a year on the information and experience available, and expressed as a cost per 1,000 A.M.'s of production at the target rate for the full year. This budgeted cost of overheads is applied to the Allowed Minutes productive time represented by each operation in order to arrive at the total standard cost of such operation.

Table 4 is an example showing how A.M. values and cost per 1,000

TABLE 4.—PRE-CAST-CONCRETE UNIT

		£	*s.*	*d.*
Castings A.M.'s	131 A.M.'s at 600*d*/1,000		6	7
Reinforcement A.M.'s	37 A.M.'s at 470*d*/1,000		1	5
Loading A.M.'s	13 A.M.'s at 450*d*/1,000			6
General overheads	181 A.M.'s at 430*d*/1,000		6	6
Plant	181 A.M.'s at 112*d*/1,000		1	8
Mould charge	(actual first cost + maintenance charge) over day's life.			10
Reinforcement	(actual cost) raw material		2	3
Concrete	(actual cost) raw material		2	6
	Total	£1	2	3

units of work can be used for both estimating and costing, thus knitting together the work of the engineer and the accountant to the benefit of both.

The methods described in this Note have recently been applied to a variety of tasks, some of which are listed below :—

Rock drilling.
Plant maintenance.
Making pre-cast-concrete units.
Electric-arc and flash-butt welding.
Carpentry and joinery.
Smithing.
Switch-and-crossing manufacture.
Permanent-way stores handling.
Structural steel fabrications.
Foundry chair-moulding.
Reclamation of recovered permanent way materials.

In every case results have shown that even after due allowances have been made for increased administration charges the financial savings to the Railway Executive have been at least equal to the additional payments made to the operators.

Experience has shown that the more difficult the subject is to study and bring into the field of incentive working the greater is the financial saving to be realized.

The Author wishes to express his thanks to Mr J. C. L. Train, M.C., M.I.C.E., member of the Railway Executive, for permission to present this Note and to Mr F. E. Campion, M.I.C.E., Civil Engineer, Southern Region, and the several members of his staff who have assisted the Author with encouragement and advice during its preparation.

Introductory Note No. 2

" Economy through Organization "

by

JOHN TAYLOR THOMPSON, M.C., M.I.C.E.

This Note is one of four which deal with the methods of effecting economy in railway civil engineering work, and is concerned with economy through organization. Economy for this purpose may be taken as attaining the desired result with the minimum expenditure of labour, material, etc., and organization as the arrangements made for achieving the desired results.

In planning any organization the first essential is that a clear picture of the result desired must be obtained, whether this is for the organization as a whole or for any particular section of it. There must be a clear target if the aim is to be effective. This target, or final result, must be kept in view throughout the organization so that it is attained by the fewest possible direct steps. In fact, to parody a hackneyed railway phrase, it it desirable to ask about each step in the process of producing the result " Is this step really necessary?" Unnecessary steps are likely to be out of the direct path and to involve further steps to get back again. Each step should be a direct consequence of the preceding one and should lead directly to the following step. If, in the process of achieving the desired result, supplementary requirements are catered for, there should be a clear understanding of which are essential and which are supplementary. For instance, information for accountancy and record purposes is desirable, but it should not be allowed to interfere with the direct steps necessary for achieving the goal.

The goal of railway civil engineering is the provision and perpetuation, in good order, of railway facilities, and the work is normally and conveniently divided into New Works, Maintenance, and Renewal. It is proposed to adopt this division for the purposes of this Note since the usual sequence in the life of any facility is (1) its provision as a new work; (2) its day-to-day maintenance; and (3) its renewal as this becomes necessary, its subsequent maintenance, and so on.

NEW WORKS

New Works generally originate in a suggestion by one of the Using Departments that some facility should be provided or amended. In the first place, the request is for a rough plan with an approximate idea of the cost of the proposal so that preliminary consideration can be given to the financial case for proceeding further. If the proposed method of carrying out the project and the approximate cost are satisfactory, firm estimates and plans are called for so that authority can be sought for proceeding with the work.

The technical work involved in dealing with New Works proposals can vary between most elaborate detailed plans and estimates, and more guesses, as estimates, and free-hand sketches as plans, and most railway offices can produce evidence of very heavy expenditure involved on proposals which never proceeded beyond the suggestion stage. It is, therefore, of great importance, especially in days of technical staff shortage, that at each stage in the development of a proposal, the technical work devoted to it should bear a reasonable relation to the probability of its further development.

There are two main questions to be considered in each New Works proposal :—

(1) What form should the work take?
(2) How much will it cost?

In the past it was the practice on some Railways for the Using Department to prepare what was termed a " traffic sketch," from which the civil engineer could prepare scale plans and approximate estimates. The engineering work consisted of little more than the draughtsmanship of preparing a drawing of the idea presented in the traffic sketch, and no attempt was made to evolve the best engineering solution of the problem. Should any suggestion have been made for re-casting the plan to provide a better result, the Officer responsible for the traffic sketch would probably have regarded this as a reflection on his own proposals and, on the principle that " the customer is always right," his word would often carry the day.

It is suggested that the most economical organization for handling New Works proposals should be on the following lines. The Using Department should formulate their requirements (not the method of achieving them) in sufficient detail to present a clearly defined problem. This should be in the form of a memorandum (not a drawing or sketch unless essential to clarify the memo) and should be explained as necessary at a preliminary meeting with the civil engineer at which all the factors involved should be reviewed. It is then the engineer's business to prepare one or more solutions of the problem in the form of rough sketches sufficient only to show the practicability of the solution and to illustrate it to the Using Department. Very approximate estimates should be made at this stage with an accuracy of, say, plus or minus 20 per cent. These sketches and estimates constitute the first stage of the proposals and should be sufficient to enable a decision to be reached on whether or not a satisfactory solution of the problem has been obtained and if there is likely to be a financial case for the work. If a favourable decision is reached, general drawings will now be made and firm estimates prepared, on which a final decision about obtaining authority can be based. If it is decided to proceed with the work, it is suggested that working drawings should follow, not precede, the authorization, although this inevitably involves a longer period between the authorization and the starting of site work. The alternative, however, is to make the working drawings in anticipation of authority which may not be forthcoming. The total time for developing the proposal, up to starting work on the site, is, of course, the same in both cases.

In the case of a revenue-producing scheme there are good reasons for executing the work in the shortest possible time and increased cost can indeed be justified in order to make the expenditure productive as quickly as possible. To avoid delays during construction, especially when a number of contracts are involved, the letting of the contracts should be timed to ensure sufficient allowance for the preparation of materials and the assembly of plant.

In general, the organization for carrying out contracts for new works follows that normally employed in other civil engineering work. Programming of work so that all concerned are aware of the times at which the various sections are to be undertaken, progress records to show the position of the work at any time and to adjust the programme as may be necessary, cost records, etc., are all part of the normal procedure and do not call for special comment in this Note.

DAY-TO-DAY MAINTENANCE

Railway civil engineering maintenance involves two main sections of work—permanent way on the one hand and structures on the other—and fundamentally different methods have been adopted for dealing with these two classes of work. Since railways began, the permanent way has been maintained by stationing men in gangs at intervals along the line and making them responsible, on a specified length, for dealing with the effects on the track of traffic and weather, etc. Structures, on the other hand, are not, in general, subject to the direct effects of traffic and they have been maintained on the basis of giving attention to defects as these became apparent. The organization of these two classes of work are considered separately in the following notes :—

Permanent Way

The two factors involved in considering the economical organization of permanent way maintenance are :—

(1) The amount of work to be done.

(2) The number of men, the plant and tools, and the organization to do it.

If the amount of work can be given in a convenient unit and it is known how many units one man can do, then the necessary manpower is easily determined. The work, however, covers a wide range, including maintenance of plain line, points, and crossings, grass cutting, cleaning drains, repairing fences and roads, etc., and some means of combining these is necessary. One method is to regard the work of maintaining 1 yard of plain line as the unit of measurement and to convert all other work into terms of yards of plain line. For instance, 20 yards of plain line might be taken to equal one pair of points, or 30 yards to equal 1 acre of grass cutting. By a system of equivalents such as these, the work involved in maintaining a given length of railway, including the slopes, ditches, roads, fences, etc., can be evaluated and the problem then resolves itself into deciding how many of these units a man can maintain. To determine this, a number of well-maintained lengths are selected and the " equivalent yards " in each length calculated. Dividing this number of " equivalent yards " by the number of men employed on the length gives the " equivalent yards per man " on each of these lengths and the average may be adopted as a basis for other lengths. In this way, the manpower necessary for day-to-day maintenance on any length of track may be determined. This method has the merit of being based on simple principles but, in itself, makes no provision for varying conditions affecting the work required to achieve the same standard of maintenance. For instance, it is well known that formation conditions play a large part in determining the amount of work involved in maintaining the track, and a formula based on " manpower in relation to track mileage " takes no account of this. Special allowances can, of course, be made to cover the estimated additional work arising from abnormal conditions but the more this is done the less the " manning of the line " is based on uniform standards, and experience of requirements on each length tends to take the place of calculation.

The principle of a standard manning of the line to deal with day-to-day requirements could well be reviewed in the light of present-day conditions. Two features have arisen since the days when this principle was first adopted. The track itself has been improved in design and, whereas daily attention was essential years ago, the introduction of flat-bottom track with its stronger and stiffer section and the absence of keys, as well as the adoption of improved fastenings, make it possible for the track to go for long periods without repair. On the other hand, the mechanization of track work has raised new considerations, since the issue of power plant to small length gangs is generally uneconomic.

Whilst daily attention is required to deal with emergencies, the repairing of the track can be regarded as work which should be dealt with as it becomes necessary, as is done with structural maintenance work. On this basis, the usual length gangs responsible for fixed lengths of line would be replaced by patrolmen for day-to-day safety and by mechanized gangs carrying out specified repair work, as required, over an extended area. The economy of an organization on these lines would arise from the manpower being provided only as and when required for specified work, as compared with the provision of manpower on a standard basis of assumed requirements for a given length of line.

The organization of day-to-day track maintenance is probably the most important of all railway civil engineering economic problems at the present time, when shortage of man-power and high labour costs, mechanization and new track designs all combine to justify a reconsideration of past practice on the lines just suggested.

Structures

The day-to-day repairs involved in the maintenance of structures call for careful organization if wasteful use of labour is to be avoided. Experience has shown that a system under which general repairs are carried out at each station every year avoids, on the one hand, the undue accumulation of repair work and, on the other hand, prevents the waste involved in dealing separately with small repairs as they arise. Under this system a District is divided into six suitable sections and 2 months are devoted to the repairs in each section, the whole District being thus covered in 12 months. Stationmasters and others are advised, about 3 months before, when the general repairs are to be undertaken, and are asked to supply particulars, as far as they are aware, of the repairs that are necessary. Armed with this information the supervisory staff visit each station, etc., and take details of the repairs to be undertaken. The details are listed on a form giving particulars of labour and material for each item, which is given a reference number, so that stores or manufactured articles can be identified when sent to the site. The form enables a summary of the man-days for each trade to be made and thus the total labour required can be ascertained.

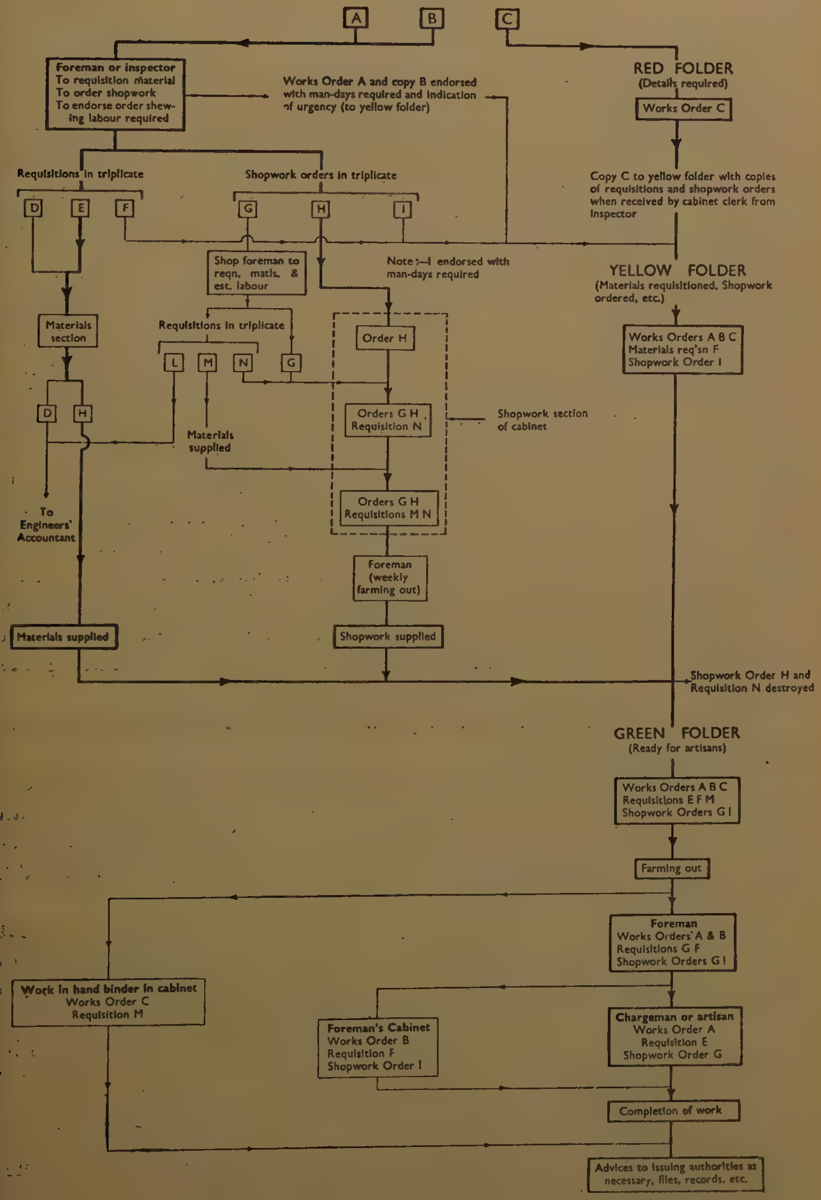
The man in charge of the work is supplied with a copy of these forms and from them he knows the items of work to be carried out, the estimated labour required at each place, and the reference numbers of the items which will be given on the materials, etc., when they are delivered.

For carrying out the work special general repair gangs are organized, including joiners, masons, plumbers, etc., as required, and these men work together all the year round, except when variation in the amount of work in repair areas necessitates adjustments in the number of men in the gangs. In order to decrease the amount of site hand-labour a travelling workshop is attached to each gang, equipped with power-driven plant.

So far as possible, small repairs are carried out during these general repair periods but, if urgent repair work arises between repair periods, it is dealt with through the "Cabinet" procedure described below, or by a local contractor, depending on the travelling time involved if carried out by railway staff. In many cases it has been found desirable to institute a "travelling plumber". arrangement under which stations, etc., are visited on a regular roster by a plumber who carries out "tool bag" repairs and reports any larger repairs needed, giving details of the materials required.

Fig. 1

WORKS ORDERS IN TRIPLICATE


A
B
C
Foreman or inspector
To requisition material
To order shopwork
To endorse order shewing labour required
Works Order A and copy B endorsed with man-days required and indication of urgency (to yellow folder)
RED FOLDER
(Details required)
Works Order C
Copy C to yellow folder with copies of requisitions and shopwork orders when received by cabinet clerk from Inspector
Requisitions in triplicate
D
E
F
Shopwork orders in triplicate
G
H
I
Shop foreman to reqn. matls. & est. labour
Note :—I endorsed with man-days required
YELLOW FOLDER
(Materials requisitioned, Shopwork ordered, etc.)
Works Orders A B C
Materials req'sn F
Shopwork Order I
Materials section
Requisitions in triplicate
L
M
N
G
Order H
Orders G H
Requisition N
Shopwork section of cabinet
D
H
Materials supplied
Orders G H
Requisitions M N
To Engineers' Accountant
Foreman (weekly farming out)
Materials supplied
Shopwork supplied
Shopwork Order H and Requisition N destroyed
GREEN FOLDER
(Ready for artisans)
Works Orders A B C
Requisitions E F M
Shopwork Orders G I
Farming out
Foreman
Works Orders A & B
Requisitions G F
Shopwork Orders G I
Work in hand binder in cabinet
Works Order C
Requisition M
Foreman's Cabinet
Works Order B
Requisition F
Shopwork Order I
Chargeman or artisan
Works Order A
Requisition E
Shopwork Order G
Completion of work
Advices to issuing authorities as necessary, files, records, etc.

Cabinet Procedure for Works Control

The Cabinet System

The " Cabinet System " is the name given to a method of dealing with works matters from inception to completion. The filing and arrangement of the necessary orders, requisitions, etc. are dealt with in a specially designed Cabinet, which gives rise to the name adopted for the system. Experience has shown that this method gives rise to economy in both labour and material.

The sequence of events in a normal work of repair is as follows: a request for the work; an inspection to ascertain the form the work should take and to obtain details of the labour and material required; the ordering of the necessary material and shopwork; the supply of these materials; the allocation of the necessary labour and time for the work; the execution of the work; and the certification by the responsible supervisor that the work has been satisfactorily completed. This is the basis of the Cabinet system of works control, and for each section of line, or type of work, three folders, red, yellow, and green, are used for the documents at the different stages of the work. The red folder contains Works Orders for which details of labour and materials are still to be obtained; the yellow folder holds the Works Orders for which these details have been obtained and the necessary requisitions and orders placed, copies of these being attached to the Works Order; and the green folder contains the Works Orders, with copies of requisitions, etc., attached, for which all material and shopwork have been supplied, and which can be put in hand as soon as labour is available. The Works Orders, therefore, pass from folder to folder as arrangements for doing the work develop and the position of any Order can be quickly ascertained by consulting the Cabinet.

The details of the procedure in passing the Order through its various stages are shown in *Fig. 1.*

It has been found convenient to deal with programme works, referred to later, as well as day-to-day work, by this procedure, which ensures that all the various steps involved in executing work are carried out systematically and that work is put in hand only when all the necessary arrangements, and supplies, have been completed.

In connexion with the day-to-day maintenance of structures, a great risk of waste lies in the danger of " finding work for the men rather than men for the work," and this tends to arise from a fixed labour force being alloted to a fixed area. To avoid this, the organization must provide for definition of the work to be done and of the labour required to do it. In addition, the amount of site work to be carried out weekly is measured not only by the men available but by the availability of the necessary stores and shopwork and it is most important to see that the manpower in the shops and on the site are balanced accordingly. In using the Cabinet system of control the amount of work represented by the Works Orders in the green folder (ready for issue) gives a measure of the labour forces required and the average level of this over an extended period indicates the regular labour force that is justified.

RENEWALS

Permanent Way

In order that re-laying work should be directed to the sections of track most in need of renewal, it is essential that it should be based on a system of inspection and allocation of priorities. From financial, material, and staff points of view, it is desirable that, so far as safety considerations permit, the amount of renewal carried out annually should be about the same, and this amount is, of course, largely determined by past experience.

The line of responsibility for the track passes up from the ganger through the Inspector to the District Engineer and the Regional Engineer, and it is natural that the recommendations and decisions on renewals should follow the same course. The following organization for dealing with renewals has been effective in achieving economies in work and man-power and is now in very general use ;—

(1) The Permanent Way Inspector makes proposals in a specified form for the renewals which he recommends, giving details of the track material and its present condition.

(2) These sections of track throughout the District are inspected by the District Engineer and marked with a degree of urgency so that the essential work can be easily distinguished from items which could be deferred. On the basis of this inspection he recommends to the Regional Engineer the programme of renewals to be carried out in his District. To ensure that all items which should be considered for inclusion in the programme have been inspected, the Chief Permanent Way Inspector is required to give a certificate that all track which, to his knowledge, should be considered for renewal has been submitted for inspection.

(3) The Regional Engineer, or one of his assistants, inspects the proposed renewals submitted by the District Engineers (either the whole or selections) and determines the order of urgency for the Region as a whole. The most urgent of these, to the extent of the annual renewal mileage, form the recommended programme, provided, of course, that no question of safety is involved in any of the items excluded on this basis.

(4) Once the programme has been authorized, the main material supplies (rails, sleepers, and timbers) are arranged so that deliveries will cover the work throughout the year.

(5) Since track renewals necessitate possession of the line and speed restrictions, the order and times of the renewals are, at least for important lines, agreed with the Operating Department

and a programme drawn up giving the dates when the various items will be carried out.

(6) This programme is arranged to show the renewal gang, or gangs for the work, and the date for each item, and is reviewed from time to time in the light of the progress made.

(7) Drawings for junction work, and the manufacture of the necessary points and crossings, are arranged to fit in with the time-table mentioned under (5).

Structures

The principle of inspection and priority mentioned in connexion with permanent way applies equally to structures. Regular inspection and reports by works inspectors and bridge examiners are made and, on the basis of these and his own knowledge, proposals are submitted to the District Engineer by the works inspectors and the Chief Works Inspector. Following the same procedure as outlined for permanent way, the District Engineer makes inspections to ascertain the most urgent works in the District, and the Regional Engineer, or his assistants, follow this with inspections to place the District proposals in Regional priority.

In the case of bridge renewals, proposals for renewal may arise either from the physical state of the structure or from the stresses revealed by calculation. If the latter is the determining factor, the calculated stresses should be checked by tests, as in most bridges it is found that the actual stresses under working conditions are less than those indicated by calculation.

Proposals for large works which involve considerable investigation and preparation of drawings should, of course, be made as long as possible in advance of the time when the work should be carried out.

As soon as the amount of work to be included in the annual programme has been determined on the basis of the inspections mentioned above and the financial provision for the year, the works are arranged in a time-table, taking into consideration the preparation of drawings, requisitioning of material, availability of manpower at the requisite time, speed restrictions, and the hours of daylight and seasons of the year, so that indoor work should, so far as possible, be carried out during the winter months.

Progress Records

In order that a general picture of the progress being made on the various programmes during the year may be obtained, monthly returns of programme progress are submitted to the Regional Engineer by the District Engineers, and to compare with these, the Engineer's Accountant supplies corresponding figures on the expenditure incurred. In this way, the progress of work and expenditure throughout the year are easily compared with the rate necessary to complete the work in 12 months.

To ascertain the percentage progress of a programme consisting of a number of items, all in different stages, including some not started and some completed, the sum of the percentage progress of each item multiplied by its estimated cost, is taken as a percentage of the total estimated cost of the programme.

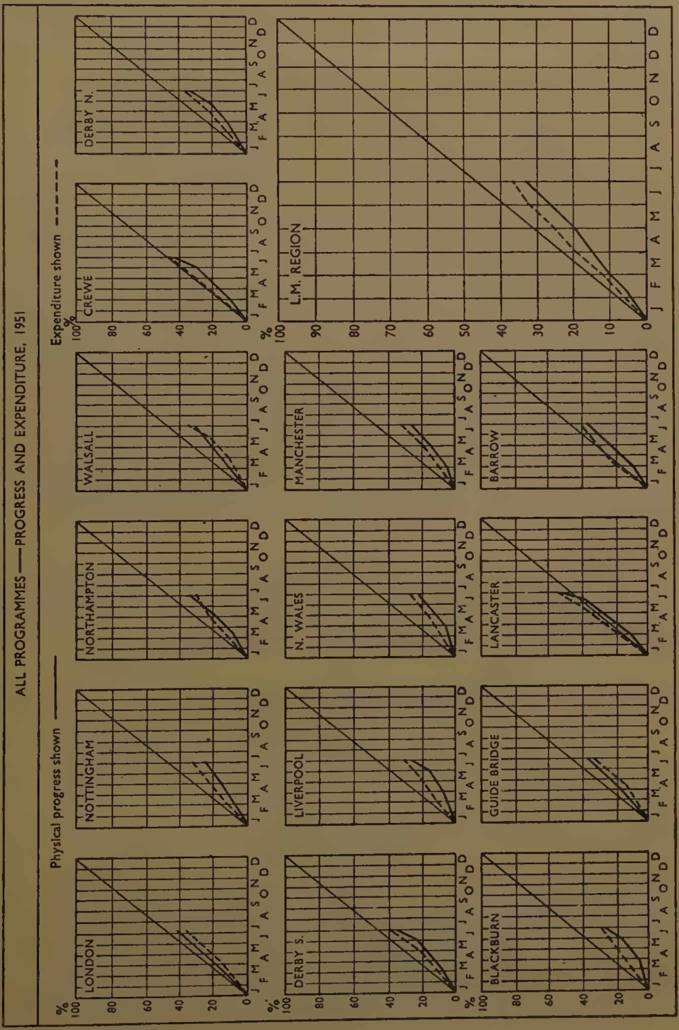


Fig. 2

ALL PROGRAMMES—PROGRESS AND EXPENDITURE, 1951

Fig. 2 shows an example of how these percentage progress and expenditure figures can be presented for Districts and the Region as a whole so as to give a clear overall view, in graph form, of the position.

ORGANIZATION AND MANAGEMENT

These very general notes on economy in railway engineering through organization refer to the economy to be effected by the adoption of a system of working that eliminates, so far as possible, waste of manpower or material by ensuring that the provision made corresponds with the ascertained need.

They refer to the economy arising, not from good management but from the form of machinery through which management operates. Management, in this sense, is not organization—the latter may be likened to the machine and the former to the operator of the machine. In other words, organization is inhuman but management very human, and that is why, especially in very large concerns, the greatest weakness is likely to arise from management rather than organization. As to the relative importance of the two, it is fairly safe to say that good management of a poor organization may produce better results than poor management of a good organization—just as a good workman with a poor tool may produce a better result than a poor workman with a good tool. For the highest efficiency and economy a good organization must be operated by an able and vigorous management and this is of particular importance in railway engineering where those exercising management functions are spread over large areas as compared with the concentrated management handling a factory organization.

Introductory Note No. 3

" Economy by Supervision "

by

ALEXANDER KEY TERRIS, B.Sc.(Eng.), A.M.I.C.E.

INTRODUCTION

The fundamental problem of industry at the present time is how to achieve the greatest efficiency in the use of manpower, and in view of present-day labour shortages this problem is of special importance to the Civil Engineering Department of British Railways.

The British Transport Commission's published accounts for 1950 show that expenditure on the maintenance and renewal of way and structures amounted to £47,000,000. Since under existing conditions the labour element accounts for about 60 per cent of the total, it follows that the wages proportion was in the region of £28,000,000, and this figure gives some indication of the civil engineering expenditure which can be controlled by management through direct supervision.

It is obvious that the problem deserves earnest consideration in order to obtain the maximum effort for the huge sums expended by the Department in the employment of labour. Basically, it is one of assessing the quantity and quality of supervision which it is necessary to apply to any one task or series of tasks in order to achieve the highest degree of efficiency.

Three fundamentals—organization, inspection, and supervision—form in varying proportions the constituent parts of all administrative work from the highest to the lowest level. The emphasis which is placed on any one of these fundamentals at any and every level, theoretically is dependent upon so many factors that it is incalculable. In any organization the quantum of supervision is usually built up from experience and must be continually under review, since it is concerned with human beings with their infinite variety of individual and collective characteristics, and also because of the necessity to match it with changing order and circumstance. It is therefore hardly surprising that there appear to have been few attempts to assess the efficiency of management, or its component part supervision, in general terms, and each organization, large or small, must be a study in itself, taking into account the characteristics of the local labour, the proportion of skilled to unskilled workers, the extent to which the labour forces are dispersed (which will be referred to as the dispersal factor), and the degree to which the task lends itself to repetition or planning on a time basis.

Large sums of money can be wasted by maladministration or faulty supervision of work units consisting of gangs of men which are common in railway civil engineering work. The repeated loss of a comparatively small element of time or of effective effort by a gang of, say, twenty-five to thirty-five men can very quickly justify the cost of a first-rate supervisor and the staff required to plan the work effectively beforehand in the Drawing Office or Works Control Section.

DEFINITION

Supervision is concerned, in varying degree, with :—

(1) Detailed assessment and planning.
(2) The quantum performed by the individual.
(3) A satisfactory standard in the work executed.
(4) The economic use of materials and plant.
(5) The recording of progress and costing.

In the broadest sense, when organization and setting of standards of quality are called for, the word " management " is more appropriate than " supervision " in describing the function. In its narrowest sense it is generally the overseeing of a particular work and the supervisor's job is to ensure that those for whom he is responsible carry out their appointed task.

In the workshop, the problem of supervision is at its simplest. The dispersal factor is reduced to the minimum, the industry of the workers ensured to a large extent by piecework or bonus schemes, and the level of output of each individual or of a unit is more readily available for comparative purposes. Moreover, in the case of manufacturing processes it is usually possible and convenient to segregate the function of inspection for quality, and the foreman or primary supervisor's responsibility can be confined to the maintenance of an economic output in terms of quantity.

APPLICATION TO RAILWAY CIVIL ENGINEERING

The bulk of the railway civil engineer's expenditure is on large numbers of comparatively small renewals or reconstructions, general repairs, and day-to-day maintenance to a diversity of structures such as permanent way, buildings, bridges, earthworks, fences, drainage, roads, water-supplies, etc., dispersed over a wide area. Continuous site supervision is in general impracticable and difficulty arises in providing standards of output on which the measurement of efficiency can be based. In fact, the standard is usually that of quality and safety.

The term supervision, used in its broadest connotation, embraces at district level any man who is in charge of others and who is responsible for the overseeing of their work from a chargehand craftsman or permanent-way ganger to the District Engineer. For the purpose of this Note, the term has been restricted to those above the level of chargehands and gangers and below the District Officer and his Assistant, and is confined to those who are known as Inspectors.

Normally the Inspectorate in a District Engineer's organization is divided into two sections, namely those in charge of permanent-way staff and those in charge of workshops staff, the former being sub-divided between maintenance staff and re-layers.

There are other fields of supervision highly essential in any railway undertaking, namely, those concerned with the custody, care, and distribution of materials and plant and of manufacturing depots, but limitations of space will not permit in this Note detailed consideration of the practices involved.

MAINTENANCE OF PERMANENT WAY

Maintenance of the track, earthworks, drains, and lineside fences is mainly carried out by gangs of five to seven men under a length ganger. Normally, the length ganger arranges the day-to-day work of the gang, subject to the guidance of the Inspector, and in this he is assisted by a calendar of systematic maintenance work to be undertaken throughout the year.

The Section Inspector is, however, not only the supervisor of the work on the length gangs; he is also responsible for the safety of the line and for the original assessment of renewals required for the following year's programme, and is in consequence involved in a great deal of inspection work in addition to being concerned with the work done by the gangs. Moreover, it is obvious that with normal length maintenance by gangs the ratio of supervisors to workmen must be largely governed by the geographical length which can be covered by the supervisor in his capacity as an inspector.

The Section Inspector has normally a fixed labour force and his use of material is relatively small, so that his efficiency and that of his men is for the most part reflected in the returns obtained by the marking-of-lengths system.

On the Eastern Region the number of lengthmen to one Section Inspector varies from 62 to 102, the average being 80. Although this proportion is high relative to outside civil engineering practice, the arrangement has operated efficiently over a long period of years. It may be that the low factor of supervision in this field has been possible by the fact that in the past a good type of labour has been attracted to the railways.

Unfortunately, this high standard of labour is not always available and, in consequence, the Section Permanent Way Inspector's responsibilities are becoming increasingly onerous.

Renewal of Permanent Way

Continuous and adequate supervision is necessary in this category of work in order to ensure the maximum amount of work in the least possible time, thereby reducing the time of line-possessions, decreasing the period of operation of speed restrictions, and carrying out the work at the minimum cost.

On the Eastern Region a Re-laying Inspector is allocated to each District and he is responsible under the Chief Permanent Way Inspector for the planning and supervision of the work. Sub-Re-laying Inspectors are provided on the larger Districts. Gangs consist of twenty-five to thirty-five men, with a Ganger and Sub-Ganger. It is the practice for an Inspector to be in charge of each job on Sundays and the low average cost of re-laying in the region would indicate that this arrangement is satisfactory. When the number of men engaged is greater than fifty or sixty, an additional supervisor is considered to be justified.

Maintenance of Bridges, Buildings, Etc.

The maintenance of structures is usually undertaken from the District Engineer's central workshops, the execution of a large number and great variety of works being involved, often at sites remote from the men's home station. The most careful planning of such works is necessary to avoid unnecessary travelling time and wastage of labour and much can be done by pre-assembly methods in the workshops.

There is no doubt that a measure of the success of the Cabinet System of works control arises from the concentration of labour under effective supervision which is possible by the systematic painting and repair periods for designated zones. The number of men who are required to travel, at one time, to work in comparatively scattered places, with correspondingly weak supervision, is reduced to the minimum and this is of growing importance in view of the reduced basic hours of work and standards of performance.

Supervision by Organization

In the ascending levels of supervision in railway work, the numbers of supervisors at a particular level diminishes with extreme rapidity and the tree of organization immediately is topped by the single person of the District Engineer who is responsible for management of his district.

On the Eastern Region the number of Inspectors in the District Engineer's organization varies between eight and fifteen on the permanent-way side and seven and twenty on the works side, and it has been found essential to interpose in each case Chief Inspectors. These Chief Inspectors have responsibility for the general organization of work, allocation of labour, and the attainment of standards both in maintenance and renewal works.

It is apparent that one man could not supervise effectively a unit of the size of the average district, without some form of organization which will provide, to the greatest possible extent, a direct and visual control of the actions of the primary supervisors or inspectors. This can be accomplished through systems of control over work which should be as simple as possible but which, at the same time, provide current information to the District Engineer and his Chief Inspectors on such matters as labour and materials, engine hire, progress of works, and expenditure.

Supervision of Works

The constant co-ordination which is essential between the various sections of building, carpentry, steelwork, plumbing, and painting, on the works side clearly points to the need for a Chief Supervisor at a somewhat higher level than the Works Inspectors.

The question of the trade of the Inspectors under the Chief Works Inspector and the desirability of their assuming responsibility outside their own trade in their capacity of supervisors is an important one.

Most day-to-day and programme work on the railway, apart from plumbing and steelwork in some of their more involved aspects, is within the powers of a competent inspector.

If the supervisor is appointed for the purpose of general supervision over a section of district, the points in favour can be listed as follows :—

(1) Most maintenance work involves more than one trade and the division of responsibility between two or more supervisors on a particular job leads to delays and waste of time, and lessens the responsibility of co-ordination between the various trades.
(2) The delegation of work and apportionment of labour is simplified through a works control system by such appointment.
(3) A general inspector is able to cover a more compact section of a district, so avoiding excess travelling time which would be incurred by the use of supervisors confined to one trade.
(4) The appointment of general inspectors offers a wider field of choice of the right type of man.
(5) Better provision is made for training men who will ultimately occupy Chief Works Inspector's positions.

On the other hand there is a school of thought which inclines towards supervision by trades on the grounds that the supervisor will be more " expert " in his function, but it is doubtful if, on the grounds of overall economy, the advantages outweigh the disadvantages on railway work.

Supervision of Permanent Way Work

The necessity for a Chief Inspector on the permanent-way side is not, at first sight, so apparent as on the works side.

The nature of track maintenance work and the duties of Section Permanent Way Inspectors are similar in all Sections. Although Permanent Way Inspectors are responsible directly to the District Engineers for the satisfactory upkeep of their Sections, it is clearly impossible, on account of the number of Sections, for a District Engineer, to do more than pay an occasional visit to each. To ensure that the work is satisfactory, and that suitable standards are maintained, a Chief Permanent Way Inspector, to whom the District Engineer can delegate a measure of his responsibility and his duty of frequent inspection, is found to be essential in most Regions. The particular value of a Chief Permanent Way Inspector in regard to maintenance work lies in his being in daily touch with Inspectors and maintenance staff in some part of the District; in his competence to guide and advise them in their work by virtue of his own practical experience ; in his bringing to bear a uniform standard of judgement on the relative quality of maintenance in the various sections ; and on his being in a position to judge the relative need for special assistance in labour and in material supplies.

An equally important factor in favour of a Chief Permanent Way Inspector arises from the policy followed on some Regions in regard to the execution of renewals. For example, on sections of the former London

& North Eastern Railway, Permanent Way Inspectors in the past had their own extra gangs for re-laying and similar special work. In many cases they had the assistance of a Sub-Inspector. In more recent years a limited concentration of re-laying gangs at salient centres was brought about. These gangs are placed under the supervision of a Re-laying Inspector, assisted in some cases by a Sub-Inspector, or, on very busy Districts, two. This arrangement allows the planning of work, running of material trains, and delegation of labour to be effected to the best advantage and, apart from the economies produced, a higher overall standard of work is obtained. Just as important is the consequence that Section Permanent Way Inspectors are free to concentrate on purely maintenance work ; this results in improvement and longer life of the track.

Supervision by Personal Contact

The District Engineer depends on the primary supervisor or Inspector for the satisfactory execution of the most important work of the management, that is to say, at the actual point of application of the prescribed principles of planning and method. It is essential, therefore, that the District Engineer should impart these principles to his Supervisors with a reasonable frequency and persistence which will obviate any natural tendency on the part of the Supervisor towards complacency. District Engineers should therefore meet their Permanent Way and Works Inspectors at regular intervals for this purpose. This is particularly important for the Works Section which is more complex in many ways. Particular difficulties arise in view of the number of trades involved and the greater variety of work in this Section.

Such meetings allow supervisors the opportunity of being accepted into the organization and gaining knowledge of the aims and responsibilities of higher management. Moreover, as a result of these discussions, they learn more clearly what is required of them and gain confidence and stimulation from the experience.

Present-day developments make it more than ever necessary for the supervisor and management to obtain the confidence and collaboration of the workman. To do this it is necessary that the latter should feel he is an integral part of any scheme and that his effort and good will are to be depended upon. For this reason the supervisor must also be ready to discuss, within limits of time and place, the why and the wherefore of a job of work with his men. There is no doubt that if a man is to give the best, he must be reasonably satisfied that inefficiency and uncertainty are absent and that there is method behind the preparation and execution of the work which he is called upon to perform. For this reason organization must provide the supervisor in the first place with an orderly system whereby the various phases of his function are carried out in a proper sequence for each work and also that the benefits of experience are always available to him, either from his superiors or from his colleagues.

In the Eastern Region, Section Permanent Way Inspectors hold meetings at regular intervals with their gangers and the District Engineer and his Chief Permanent Way Inspector attend for a short time. There is no doubt that much good has resulted from such meetings. Whilst this arrangement is not equally practicable on the works side owing to the variety of trades and diversity of tasks, there is no doubt that the chargehands and tradesmen would benefit by a fuller understanding of the works organization and procedure and every opportunity should be taken to impart this knowledge. The personal influence and personality of the District Engineer must always pervade the organization and for this reason alone the size of his unit and his staff should be such as to permit this.

Requirements of the Supervisor

The Supervisor is required to assess defects in the structures or services for which he is responsible. He should be able to recommend the remedy, the method of work to be adopted, the plant, labour, and material required for its execution, and thereafter be able to supervise and control the work efficiently to completion. These functions offer the supervisor his greatest opportunity to make his contribution to the efficiency of the undertaking. The waste that could result from the incorrect measurement and ordering of materials, the misuse or neglect in use of the most suitable plant or correct labour forces, is obvious. Less obvious perhaps, but just as expensive, is the waste which may be caused by inefficient planning of the work and sequence of operations. Errors in these matters are frequently the result of overloading the supervisor, and this point is a major consideration in determining the number and distribution of supervisory staff in any organization.

The scope for economies during the actual execution of a job will, of course, vary considerably with the nature of the work and with the quality of the men engaged upon it. The response of the latter will, within certain limits, be the measure of the inspector's leadership and his judgement of their abilities.

Quality of Supervision

Experience shows that successful management is ultimately determined by the ability of the Supervisors, and the maximum efficiency of an organization is attained by the selection of first-rate primary supervisors. Under present-day conditions it is not possible for the Railways to exercise such powers of selection as will always ensure that inspectors are capable of carrying out their function without a considerable measure of supervision from the managerial level.

There is evidence, however, that the quality of the labour intake to the Railway is deteriorating, and this is, and will continue to be, reflected in the ability of the Supervisors as time goes on. The difficulty which has also been experienced for some years in recruiting qualified or otherwise suitable staff for the higher administrative levels will militate against the possibility of balancing any lowering of the standard of basic supervision by more management or organization. If this view is accepted and also that any change is to be deprecated which would have the effect of reducing the responsibility of the site supervisor more than custom has found necessary, then one of the problems most deserving of consideration is how to obtain the necessary quality of supervision.

One of the main tenets which has been generally accepted is that supervisors are born, not made, and while this may be so, in view of the difficulties which have already arisen, it is now considered essential to provide facilities for the training of individuals who have the necessary basic practical experience and show evidence of qualities of leadership. If such training is not given, there will in many instances be little, apart from the authority of his position, to differentiate the supervisor from the men under his control.

The purpose of such training should be to instil better powers of observation, to impart technical knowledge and implant new and better conceptions of methods of work, provide ideas on the handling of labour, and in general to widen the outlook of the inspector.

CONCLUSION

From the above considerations it is clear that the best and most economical work will be obtained only if supervision is adequate; it is necessary in some way to find the correct balance between senior administrative or management staff and basic or site supervision. The proportion between the two, and the relationship of the total to the numbers of men employed, must inevitably be the result of the way in which the work is organized and the individual and collective ability of the administrative staff. In fact, it cannot be purely and simply a matter of numbers alone.

Whether the systems of supervision and training which have been commented upon fulfil or are likely to fulfil the requirements of the immediate future is a question with which we must be concerned. The present method of making good the labour shortages by assistance from Contractors, together with the greater proportion of casual labour within the depleted forces of the Railway Executive, render the Supervisor's rôle much more exacting. There is already evidence that, in certain Sections where these conditions obtain, his powers are unduly extended.

If the position is to persist there would appear to be certain alternatives possible:

(1) To augment the number of supervisors to provide for the increased site supervision necessary and to deal with the additional responsibility of overseeing contract labour.

(2) To increase supervision by the introduction of qualified technical staff at an intermediate level in the District organization.

(3) To introduce practical schemes of incentive bonus payments which can be expected to lessen the amount of basic supervision necessary by the Inspector. Such schemes will nevertheless add considerably to the inspection and measurement elements and may necessitate the recruitment of additional staff for the purpose.

In the Eastern Region the average percentage of supervision to wages paid for permanent way staff is 2·63 and for workshops staff 4·8. It is not, however, possible to obtain figures which will give a measure of the quantity of basic supervision involved in any particular task, from which may be deduced true economy arising from efficient supervision. Since the variation in costs of jobs depends upon many different factors, obtaining these figures would require clerical expenditure on a scale which may well be unwarranted by the results to be achieved and, in fact, it is doubtful if the assessment is possible.

As already stated, most railway civil engineering work is widely dispersed and is not subject to continuous supervision, and in some cases the quality of work is difficult to assess. For this reason there is great difficulty in measuring output. On the ability to do so rests any control system, for control must be based upon work standards and these are relatively difficult to fix in civil engineering work. At the present time any assessment of the efficiency of basic supervision for a particular task must be arrived at by comparison with the results obtained, over a period of time on similar tasks, having regard to the conditions prevailing at the time of its conception and execution.

ACKNOWLEDGEMENTS

The Author wishes to express his thanks to the Railway Executive and to Mr J. I. Campbell, M.I.C.E., Civil Engineer of the Eastern Region, for authority to present the Note and to the members of his staff for the information which they have provided.

Introductory Note No. 4

"Economy through Design"

by

Percy Stuart Attwood Berridge, M.B.E., M.I.C.E.

Introduction

An annual expenditure of £2¾ million is incurred in the examination, repair, and renewal of bridges, tunnels, and culverts on British Railways; and, of this sum £2¼ million is spent on the examination, repair, and painting of these structures.[1] Although the cost of perfection may be prohibitive, a greater degree of permanency is to be expected in the future provided close attention is paid to details, so essential in good design, and full use is made of knowledge gained from past experience.,

Economic consideration of a structure and in particular a bridge, which is an essential part of a railway and must be maintained or reproduced so long as the line is to be kept open, involves not only the initial cost of the structure and the maintenance charges during its life but also the interest on the capital set aside for replacement in perpetuity. Many of the structures built in good sound masonry have stood the test of time and can be expected to last *ad infinitum*, but for the purposes of accountancy the average life of steel and reinforced-concrete bridges in Great Britain has, in the past, been taken as only 55 years. In this Note, attention is drawn to improvements in design which, it is hoped, will increase the life of girder bridges and, at the same time, reduce expenditure on examination, repair, and maintenance.

The Choice of Design

A Comparison of Costs of Underline Bridges

The comparative costs of different forms of construction for a double-line 40-foot-long square underline span are indicated in Tables 5 and 6. Obviously these costs would vary considerably according to site conditions, etc., but for comparison it has been assumed that these spans would be erected on trestling alongside the line and rolled in when the existing spans were removed. No account has been taken of the costs of the ballast and permanent way, trestling, or substructures, or for the removal of the old spans.

In these 40-foot spans allowance has been made for the surface preparation of all exposed steelwork, and the spans will have been given two priming coats of paint (red lead and a mixture of red lead and white lead) and

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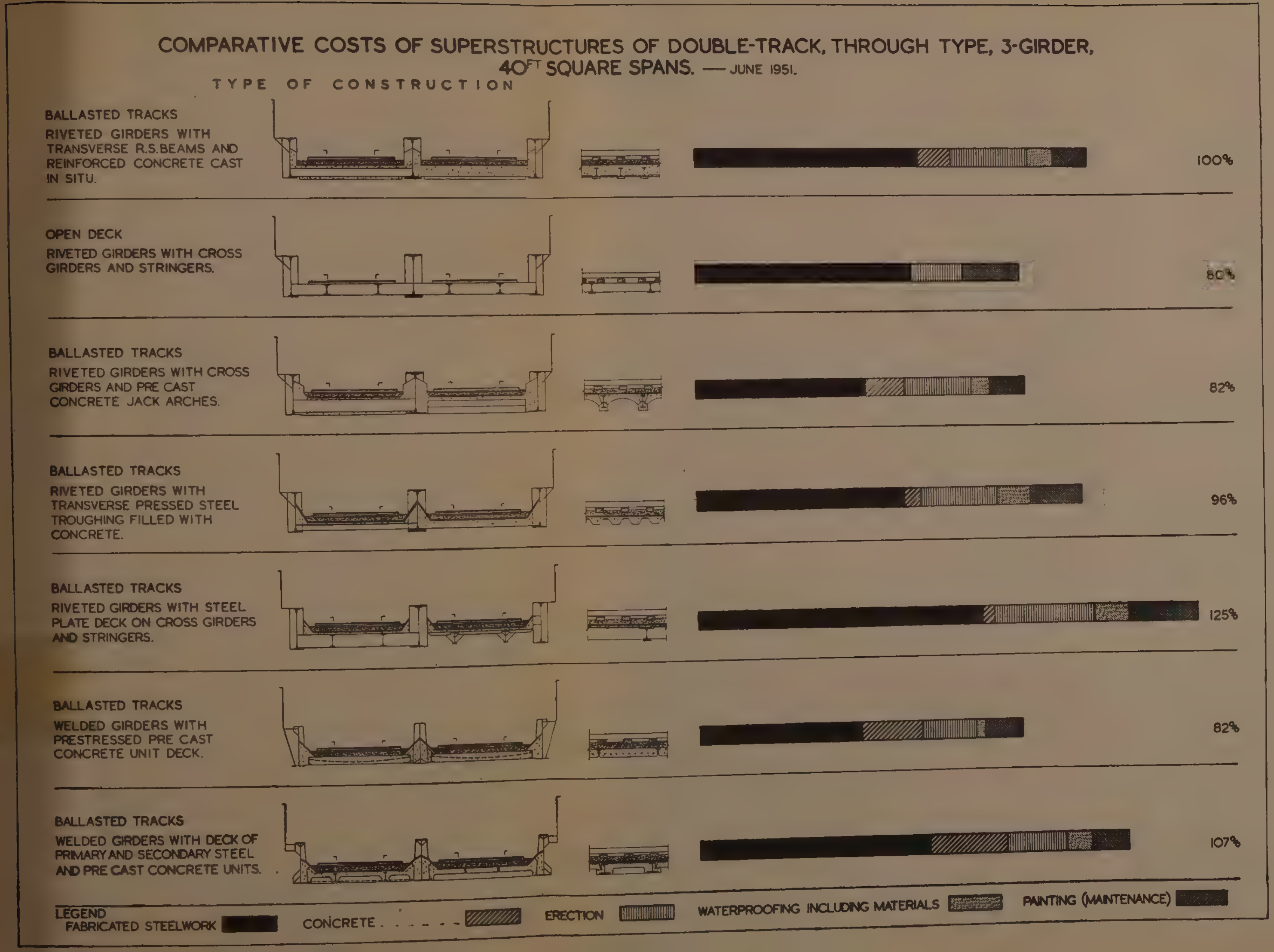
[1] The references are given on p. 142.

TABLE 5

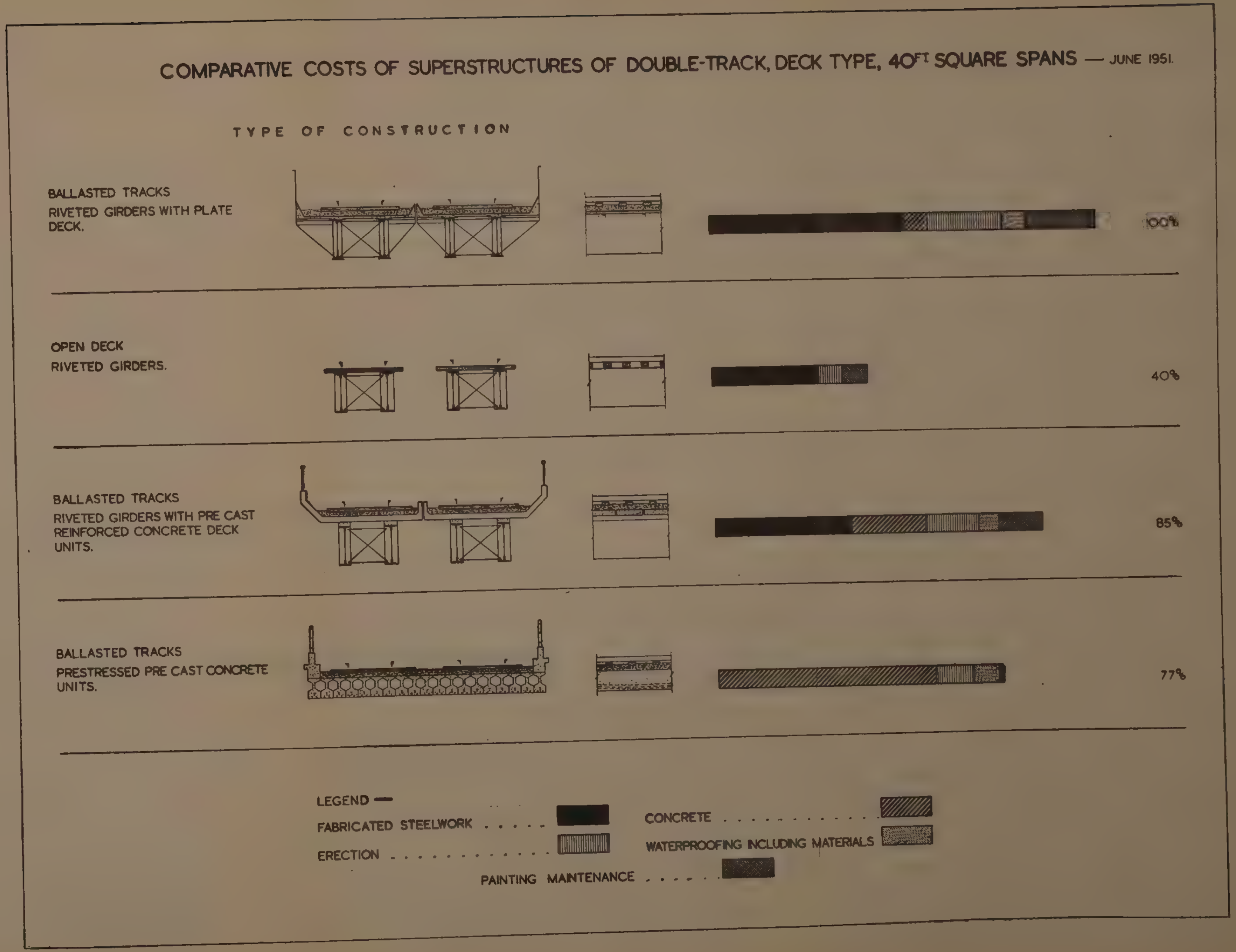


COMPARATIVE COSTS OF SUPERSTRUCTURES OF DOUBLE-TRACK, THROUGH TYPE, 3-GIRDER, 40FT SQUARE SPANS. — JUNE 1951.

TYPE OF CONSTRUCTION	
BALLASTED TRACKS RIVETED GIRDERS WITH TRANSVERSE R.S.BEAMS AND REINFORCED CONCRETE CAST IN SITU.	100%
OPEN DECK RIVETED GIRDERS WITH CROSS GIRDERS AND STRINGERS.	80%
BALLASTED TRACKS RIVETED GIRDERS WITH CROSS GIRDERS AND PRE CAST CONCRETE JACK ARCHES.	82%
BALLASTED TRACKS RIVETED GIRDERS WITH TRANSVERSE PRESSED STEEL TROUGHING FILLED WITH CONCRETE.	96%
BALLASTED TRACKS RIVETED GIRDERS WITH STEEL PLATE DECK ON CROSS GIRDERS AND STRINGERS.	125%
BALLASTED TRACKS WELDED GIRDERS WITH PRESTRESSED PRE CAST CONCRETE UNIT DECK.	82%
BALLASTED TRACKS WELDED GIRDERS WITH DECK OF PRIMARY AND SECONDARY STEEL AND PRE CAST CONCRETE UNITS.	107%

LEGEND
FABRICATED STEELWORK ■ CONCRETE ▨ ERECTION ▥ WATERPROOFING INCLUDING MATERIALS ▦ PAINTING (MAINTENANCE) ▩

TABLE 6

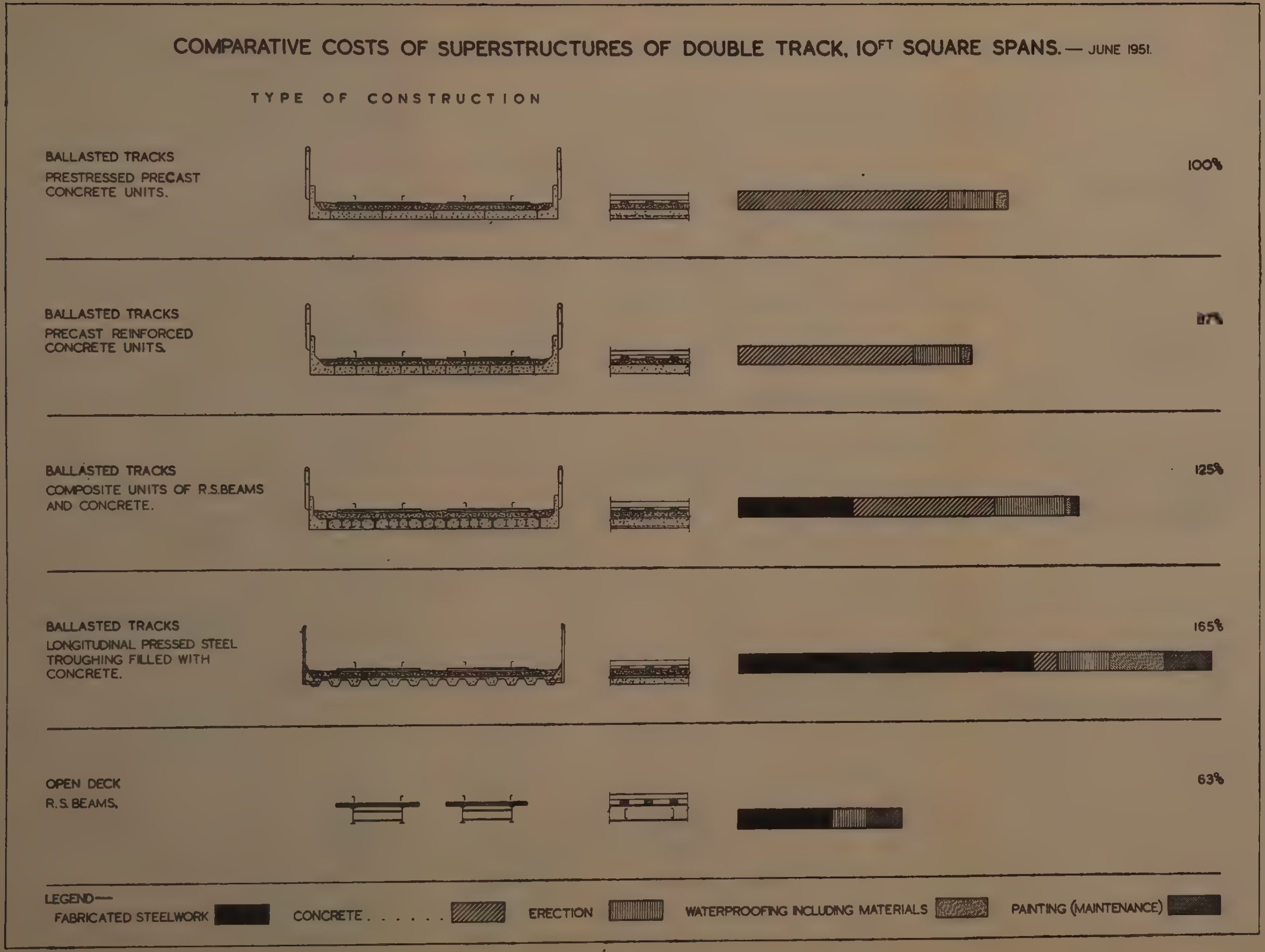


COMPARATIVE COSTS OF SUPERSTRUCTURES OF DOUBLE-TRACK, DECK TYPE, 40FT SQUARE SPANS — JUNE 1951.

TYPE OF CONSTRUCTION	
BALLASTED TRACKS RIVETED GIRDERS WITH PLATE DECK.	100%
OPEN DECK RIVETED GIRDERS.	40%
BALLASTED TRACKS RIVETED GIRDERS WITH PRE CAST REINFORCED CONCRETE DECK UNITS.	85%
BALLASTED TRACKS PRESTRESSED PRE CAST CONCRETE UNITS.	77%

LEGEND —
FABRICATED STEELWORK . . . . . ■
ERECTION . . . . ▥
CONCRETE . . . . ▨
WATERPROOFING INCLUDING MATERIALS ▦
PAINTING MAINTENANCE . . . . ▩

TABLE 7


COMPARATIVE COSTS OF SUPERSTRUCTURES OF DOUBLE TRACK, 10FT SQUARE SPANS. — JUNE 1951.
TYPE OF CONSTRUCTION
BALLASTED TRACKS
PRESTRESSED PRECAST CONCRETE UNITS.
100%
BALLASTED TRACKS
PRECAST REINFORCED CONCRETE UNITS.
87%
BALLASTED TRACKS
COMPOSITE UNITS OF R.S.BEAMS AND CONCRETE.
125%
BALLASTED TRACKS
LONGITUDINAL PRESSED STEEL TROUGHING FILLED WITH CONCRETE.
165%
OPEN DECK
R.S. BEAMS.
63%
LEGEND—
FABRICATED STEELWORK
CONCRETE . . . . . .
ERECTION
WATERPROOFING INCLUDING MATERIALS
PAINTING (MAINTENANCE)

two protective coats of paint (an undercoat and a finishing coat of micaceous iron-ore paint), before rolling in. The costs of painting during a life of 55 years cover repainting (protective coats only) after the first 10 years and thereafter at 5-yearly intervals.

The Regulations of the Ministry of Transport prohibit the adoption of the open-deck type of construction in Great Britain; but it may be asked whether, in view of the pressing need for economy, this, the cheapest form of construction, should not be adopted on branch lines on British Railways.

The trend of design is away from those types of construction where the routine painting calls for extensive scaffolding. And this comparison of costs does indicate how the latest designs of girder bridges, notably those with concrete decks, will effect economy in this direction.

The pre-stressed concrete deck-span has been included for comparison. It is, however, unlikely that such a span, made up of pre-stressed pre-cast units, brought separately to site and post-tensioned after assembly, would be erected by rolling in.

Short Spans

Similar costs of the superstructures for different forms of construction for a double-line 10-foot-long square underline span are compared in Table 7. These spans would be lifted into position by cranes and, as in the 40-foot spans, no allowance has been made for any permanent way or substructure work or for the removal of the old spans during reconstruction.

The Type of Bridge

The type of bridge is almost certain to be dictated by the construction depth available and the erection scheme to be adopted having regard to the density of traffic and the topographical features of the site.

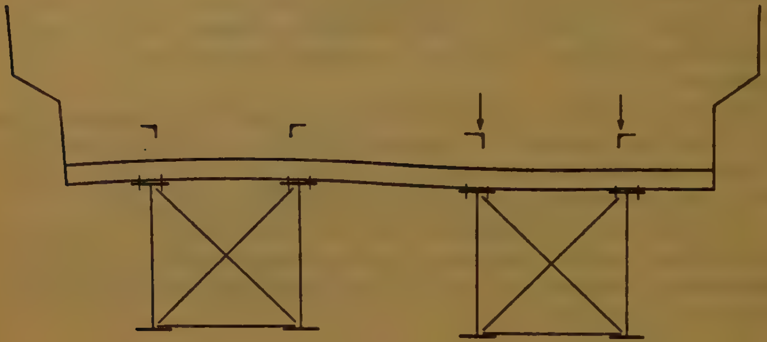
Where conditions permit, the span should be of the deck type. This involves less steelwork, is cheaper to fabricate and erect and is easier to paint because the whole of the steelwork can be readily accessible without regard to the passage of trains overhead.

Whenever possible in the case of medium-length multiple-track spans, separate structures should be provided under each track. A span which is common to more than one track suffers ill-effects from deformation stresses when the girders under a loaded track deflect and the adjacent un-loaded ones remain level. The effect of carrying the decking continuously across more than one pair of main girders in a deck-type bridge is shown in *Fig. 3.*

Deck-Type Spans

In the case of deck spans, it is important to avoid rigidly connecting a continuous deck to the tops of the main girders unless that deck shares in carrying the main bending stresses and the connexions are designed for the shear stresses imposed upon them by reason of their position above the neutral axis (see *Fig. 4*).

Fig. 3

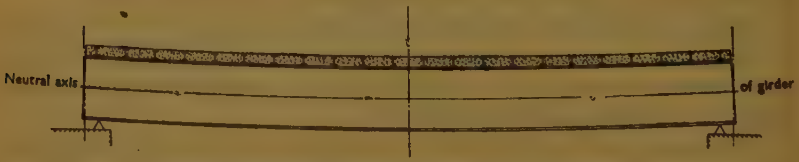


Transverse Decking Overstrained by Unequal Deflexion

Through-Type Spans

The commonest type of underbridge on British Railways is the through-type plate-girder span ; in this type, in the connexion between the decking and the main girders—particularly at the intermediate girder of a multi-track span—disregard for the first principles of design has unfortunately given the maintenance engineer a costly legacy. Space between tracks seldom permits separate spans with two main girders for each track, though such an arrangement would be the ideal. B.S. 153 states that cross-girders shall be designed as simply-supported beams spanning between the centres of main girders. In truss spans this condition can be achieved (*Figs 5*). But cross-girders landed on the top flanges of chords as shown in *Fig. 6* pivot about the inner edge of the chord and it is impossible to keep the rivets tight through the outer flange of the chord.

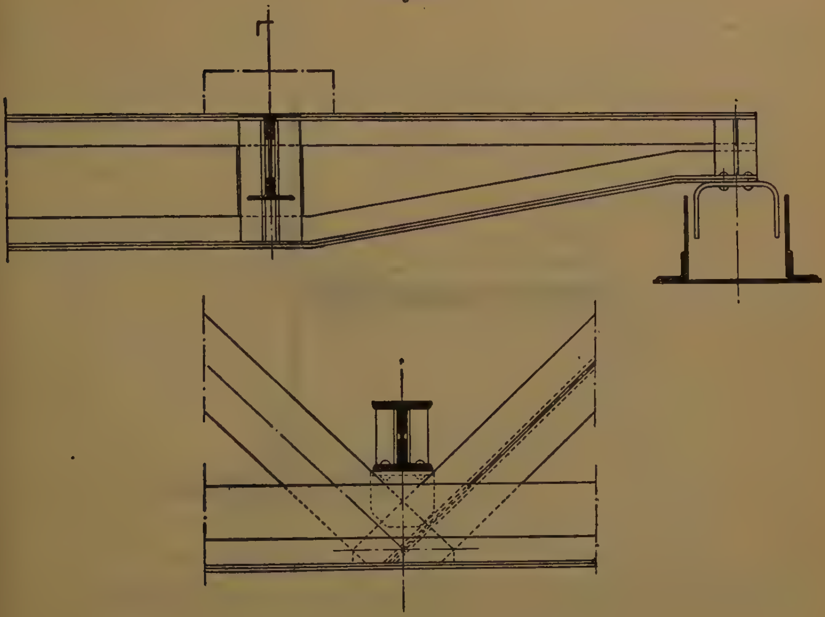
Fig. 4



Shear between Decking and Main Girders

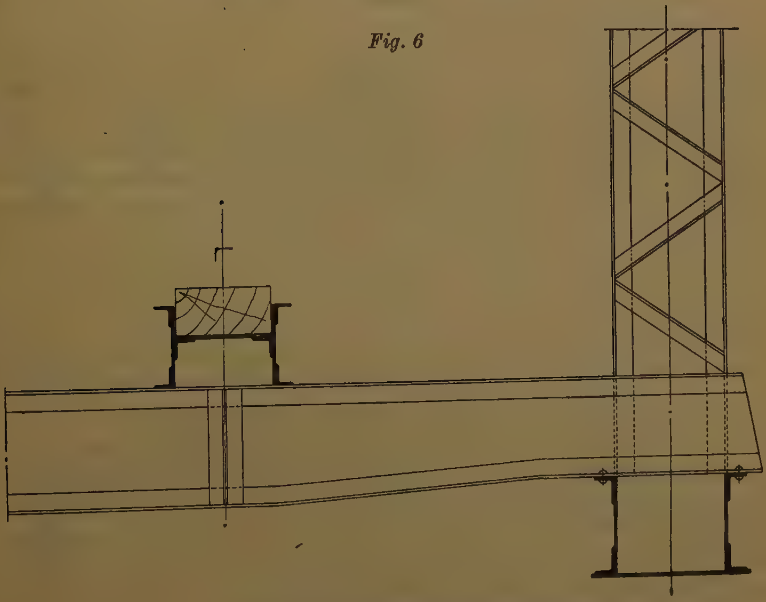
With plate-girders it is usual to cleat the decking to the main girders which immediately produces a degree of end-fixity and induces stresses

Figs 5



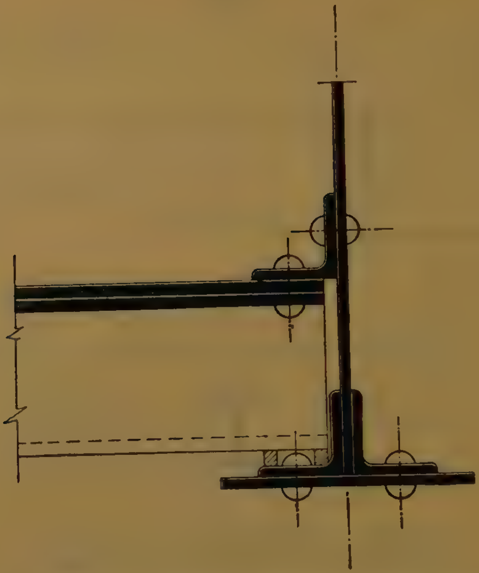
A SIMPLY SUPPORTED CROSS-GIRDER

Fig. 6



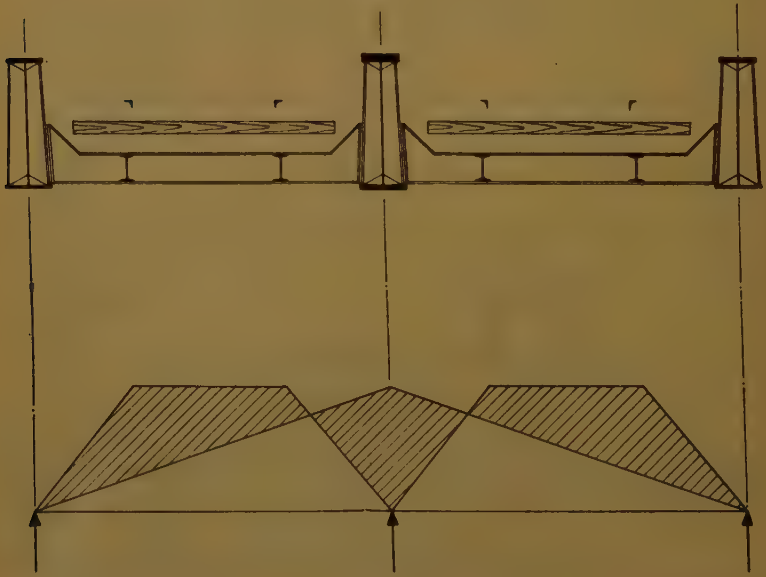
A CROSS-GIRDER WITH PARTIAL END-FIXITY

Fig. 7



Troughing resting freely on Girder Flange and rigidly connected at the Top only

Figs 8

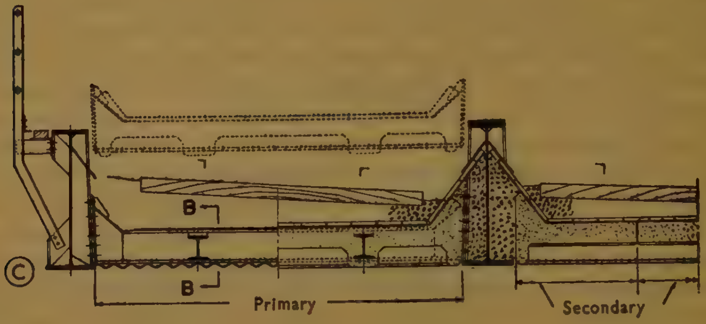


Bending-Moment Diagram for Cross-Girders. Both Tracks Loaded which the connexion is all too often incapable of carrying without overstrain. In the past some flexibility has been given by leaving the bottom of transverse troughing seated freely on the flange-angle of the main girder and lightly cleating the top of the trough to the web of the main girder. This has led to a "working" of the ends of the troughs with accelerated corrosion at a surface inaccessible to the paint-brush and to the early loosening of rivets in the cleats (*Fig. 7*). The modern tendency is to make the connexions between the decking rigid and capable of carrying the whole of the bending-moment stresses induced at an intermediate main girder when adjacent tracks are fully loaded. *Fig. 8* shows the bending-moment diagram for cross-girders considered as continuous across a centre support. A decking of primary units, with cross-girders connected with high-strength bolts to stiffener-brackets and secondary longitudinal reinforced-concrete slab units, has been designed to meet these conditions (*Figs 9*). The fixity constrains the lateral displacement of the top flanges of the main girders and allows a length of only three panels to be taken as the effective length of the compression flanges. The design includes shear plates on which the primary deck units can be landed during erection without the main girders having to be moved. The joints between the primary and secondary units permit slight movement, thus avoiding any direct stressing in the deck which is necessarily below the neutral axis of the main girders. The girderwork of the deck is encased in concrete with the object of eliminating the need for painting the underside; but unless the cross-girders are pre-stressed while the concrete is being cast around them some cracking of this protective coating must be expected.

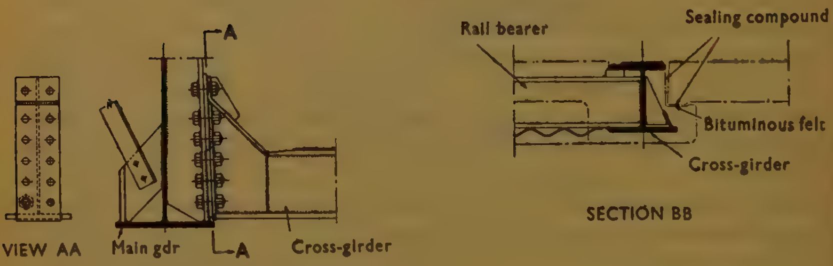
Every span irrespective of type should be absolutely self-contained, the only contact between the steelwork and its supports, piers, abutments, etc., being through the bearings under the main girders. Where stringers are employed they should not on any account rest on the masonry. Like all decking extending beyond the last cross-girder, they should be carried on a trimmer-girder connected to the main girders. No matter how short the span, provision should be made for change of slope at the bearings. Stepped bearings, permitting freedom of movement sufficient to meet alterations in length due to temperature changes, ensure freedom from damage to bedstones, and localize the points of main reaction. The old system of flat bearing-plates free to move on the bedstones has led to costly repairs to bedstones and often, by bringing the reaction to the face of the abutment, has caused tilting of the masonry.

A recent design shown in *Figs 10* incorporates pre-cast pre-stressed concrete deck units spanning between welded plate-girders. The pre-stressing bars are anchored to steel channels which, reinforced with thickening plates welded to the webs, form the ends of the units adjacent to the main girders. The site connexions between the deck units and the main girders are made with high strength-bolts passing through angle cleats, welded to the channels near their ends and to the tables of the tee-

Figs 9

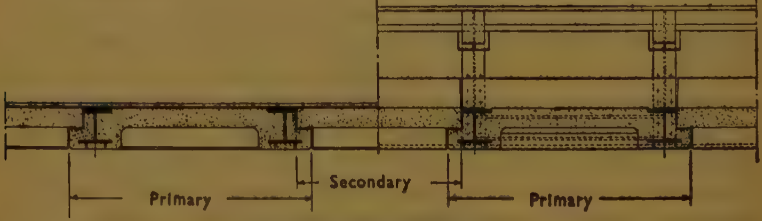


CROSS-SECTION



SECTION BB

DETAIL AT C



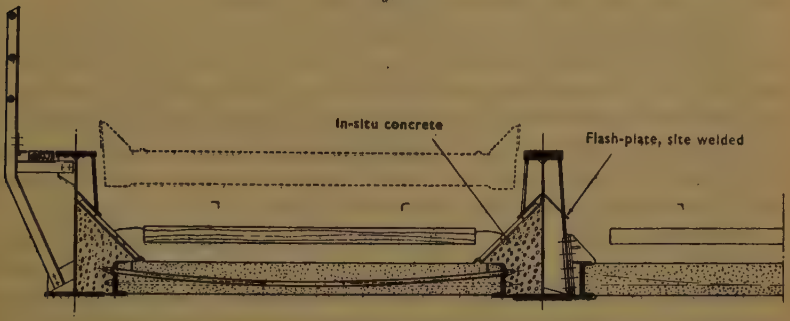
LONGITUDINAL SECTION

A Span with Decking of Primary and Secondary Units

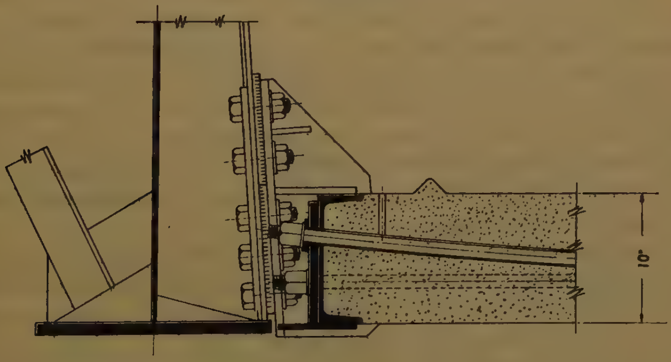
shaped stiffeners on the main girders. Shear plates welded to the cleats and resting on similar plates on the stiffeners support the deck units while the bolting up is in progress. The deck units and the connexions to the main girders are designed to carry the stresses due to full end-fixity of the deck units. There is no longitudinal post-tensioning of the deck. The pre-stressed-concrete deck is 10 inches thick, with a minimum of 6 inches of ballast under the sleepers, and the construction depth is only 2 feet 6 inches.

Where ample construction depth permits, cross-girders with pre-cast-concrete jack arches make a relatively economical deck, easy to erect and, of course, needing no painting on the underside. With the connexions to the main girders providing for conditions of full end-fixity, the

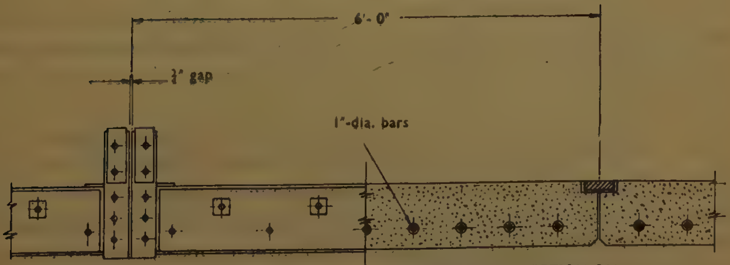
Figs 10



CROSS-SECTION



CONNEXION TO MAIN GIRDER



END ELEVATION OF DECK UNITS

LONGITUDINAL SECTION ON CENTRE-LINE OF DECK UNITS

A SPAN WITH DECKING OF PRE-CAST PRE-STRESSED CONCRETE UNITS unsupported lengths of the compression flanges of the main girders can be taken as three times the distance between cross-girders. As is usual with through-type spans, the deck may be some distance below the neutral axis of the main girders, and in consequence some movement may be expected between the jack-arch units and the cross-girders as the span deflects ; but this, it is claimed, is too slight to matter.

In a two-track bridge, three main girders are preferred to two, so that in the event of renewal of the decking, or indeed of the whole span, being necessary at a site where reconstruction by rolling-in is impracticable, the work can be carried out piecemeal without the need for total occupation of both tracks. But for longer spans where some economy in first cost can be expected, the omission of the central main girder, and so half the cross-girder connexions, will reduce maintenance costs.

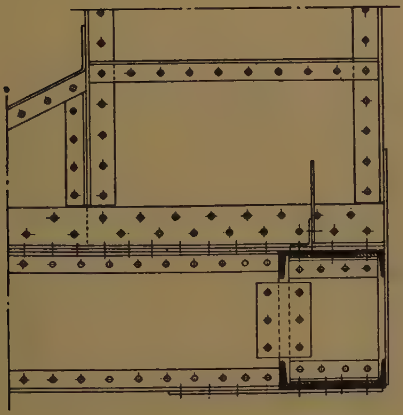
On British Railways, it is the practice to use shop-completed plate girders for spans up to about 110 feet. They are claimed to be easier to erect and to maintain than triangulated girders ; but the latter will, in a span of 110 feet, show a 10-per-cent saving in weight of steelwork.

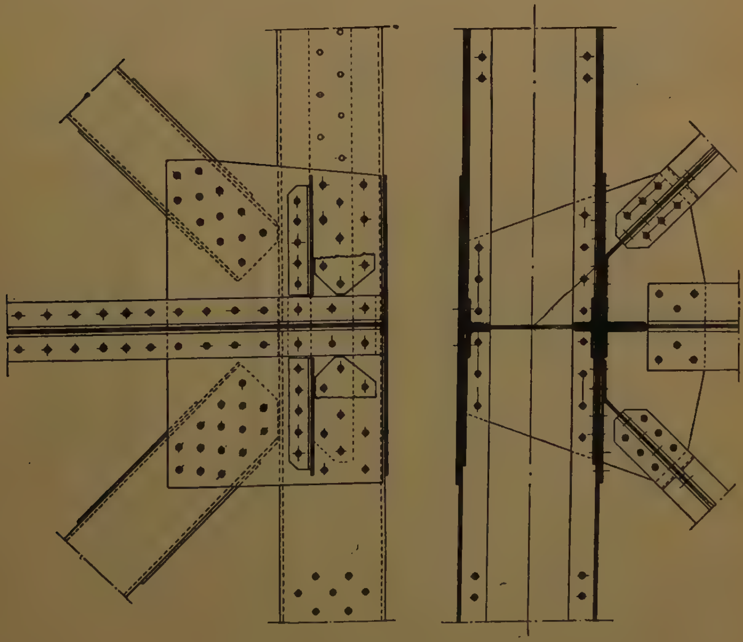
Nowadays, the single-system Warren truss with verticals is considered more economical than the Pratt truss for spans up to about 250 feet, because all the verticals will be similar. An even number of panels with a cross-girder at mid-span avoids the need for crossed diagonals. For longer spans the K-truss affords shorter stringers and requires less floor construction depth ; but weight for weight, there is little to choose between the plain and the sub-divided truss. Any advantage in favour of the latter is offset by the higher cost per ton for fabrication, due to the increased number of joints.

Some reduction in weight is gained by curving the top and bottom chords in through- and deck-span girders respectively, but the parallel chord truss is less costly to fabricate because sway-bracing connexions are always at right angles. Also, the field joints in the chords can be kept clear of panel points—a great help in assembling the girders. Underslung cross-girders connected to extended verticals below the bottom chords of the main girders should be avoided because of the very uneven distribution of the stress in the hanger ; because the bottom lateral system will almost certainly be too low ; and because of the high deformation stresses induced in the hangers through floor interaction and through traction and braking forces.

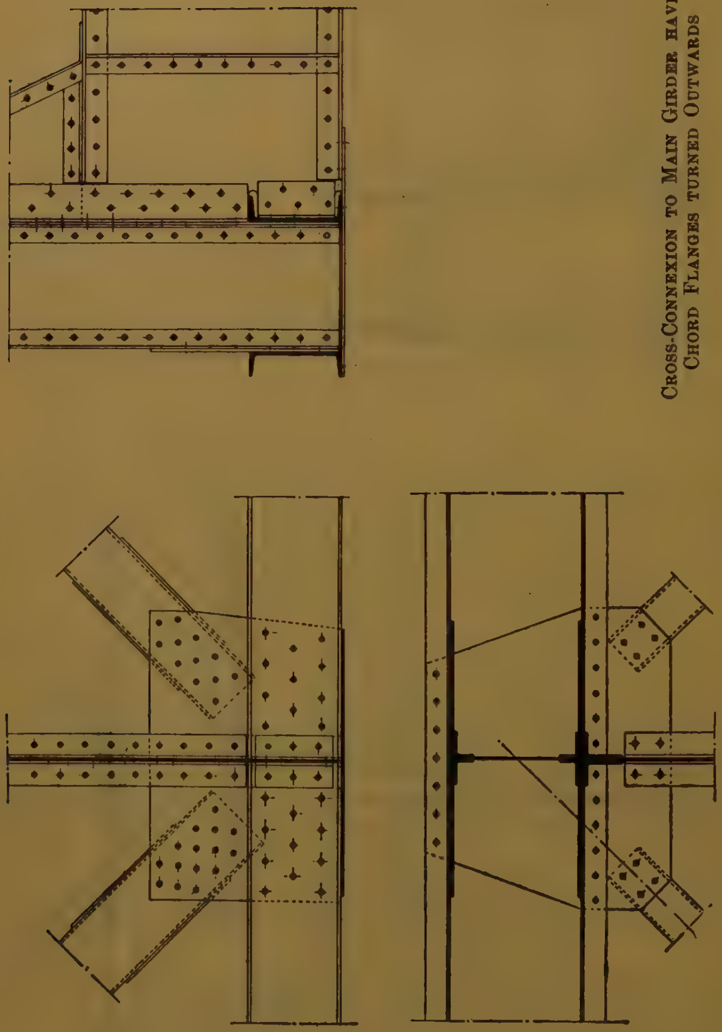
Some controversy has existed over the design of bottom chords of truss girders. The Author is of opinion that the flanges of bottom chords should be turned inwards so as to offer an unobstructed gusset-face to the cross-girder (*Figs 11*). It is much easier to fit vertical and diagonal members between the outstanding gussets than between gussets inside the chords (*Figs 12*). Additional side plates placed inside the chord webs can of course run through without break at the panel points. This arrangement enables the line of interaction of the chord to be maintained at a constant

Figs 11





Cross-Girder Connexion to Main Girder having Chord Flanges turned Inwards



Figs 12

Cross-Connexion to Main Girder having Chord Flanges turned Outwards level. For maintenance purposes the bottom flanges of chords should be open, to avoid pockets for collection of water, etc. The inward-turned flanges simplify the details at the end connexions to the rakers, the form of which should follow that of the top chords.

Continuous stringers resting on top of the cross-girders should be avoided because of the difficulty of holding down the former to the latter, the connexions to the cross-girders being seldom capable of withstanding the uplift ahead of a loaded stringer.

Floor interaction in any through-type span induces direct stresses in the stringers unless some freedom of longitudinal movement is provided for at the stringer bearings; and it is good practice to provide sliding joints at the ends of stringers on alternate cross-girders. Where this is done, lateral bracing should preferably be of the K-system, and kept quite clear of the stringers. Transverse troughing on the stringers can transmit lateral forces but a separate lateral system tying together the panel points of the floor system greatly simplifies the "squaring up" of the span during erection.

Rigid Frames

Rigid frames combined with shop-welding and site-bolting is one of the most economical constructions for short-span footbridges, station roofs, etc. It is a useful form of construction for new underbridges and overbridges, particularly where the construction depth is very limited; but it is seldom used in reconstructions because of the extensive alterations necessary to existing abutments.

Road Bridges

For the superstructure of medium spans over a double line of railway, longitudinal multiple pre-cast pre-stressed concrete beams, made on a long-line pre-tensioning system, are one of the most economical forms of construction both for square and for skew crossings (*Fig. 13*). Erection piecemeal with site concrete poured between the beams is simple and, in most reconstructions, can be arranged to give the minimum of interruption to road traffic. Steel bearing plates and bedplates with locating strips are desirable, and parapets either of open mesh or of stone or brick masonry give a more pleasing appearance than pre-cast reinforced-concrete panels. The initial cost of such a superstructure is some 18 per cent less than that of main girders with transverse steel beams and reinforced concrete, and 10 per cent less than a deck-type span of longitudinal steel girders and jack arches.

Longitudinal units of steel beams encased in concrete cost still less, but the absence of bond between the steel and the concrete, together with the risk of cracking in the concrete, make this form of construction less attractive.

Pre-cast pre-stressed post-tensioned concrete beams are eminently suitable for longer spans, but transverse post-tensioning after erection is an added complication, especially where part of the span is to carry traffic before completion of the whole width of the bridge.

Footbridges

For spans over two tracks, the welded steel rigid frame construction with pre-cast pre-stressed pre-tensioned concrete planks for the deck and stair treads, and open-mesh panels in steel frame parapets, is a very economical form of construction (*Fig. 14*). The cost is about 7 per cent less than the pre-cast reinforced-concrete bridge (*Fig. 15*). Erection is very much simpler and quicker and the appearance is more pleasing.

Costing about 4 per cent more than the rigid frame, the tee-beam pre-cast reinforced-concrete bridge with open-mesh parapets is very suitable where there are to be no stairs on the span (*Fig. 16*).

For much longer spans the stiffened arch is more economical in weight and cost of fabrication than the triangulated girder construction. In the renewal of some 100-foot spans recently, the stiffened arch girders, shop-welded and field-bolted, showed a saving in weight of 30 per cent and the cost per ton for supply was 5 per cent less than for triangulated girder spans of similar strength.

MATERIALS

Reinforced Concrete and Pre-stressed Concrete

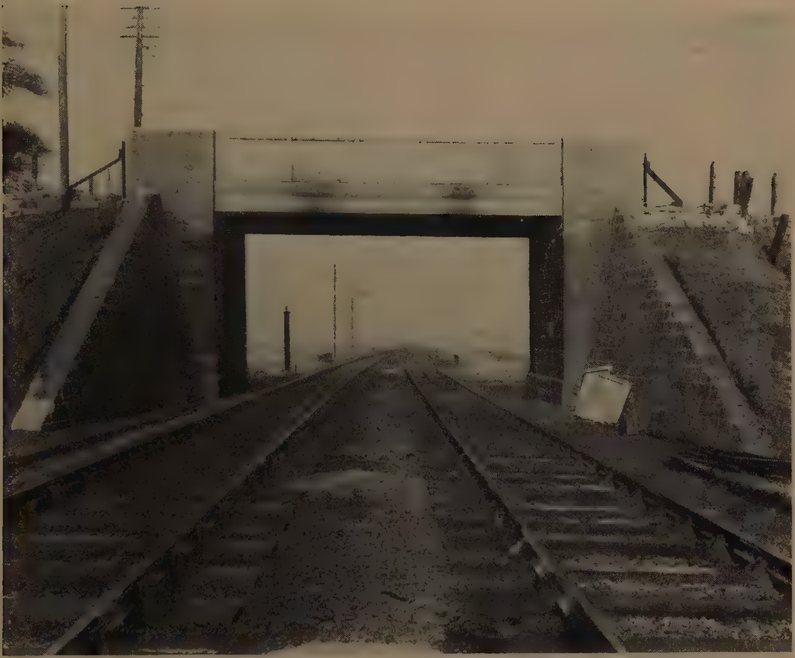
Except for the decks of underline bridges erected by rolling in, it is seldom practicable to cast reinforced-concrete bridge beams and slabs at site because the periods of occupation between trains are generally too short to allow for satisfactory curing of the concrete. Consequently, nearly all reinforced concrete and pre-stressed concrete bridge work is pre-cast.

The reinforcement near the underside of a beam suffers relatively early exposure through the cracking of the concrete in tension. The same defect exists in steel beams encased in concrete. Pre-stressed concrete, with the concrete always in compression and giving complete protection to the steel cables or bars, can, in consequence, be expected to give much longer service. Beams of lengths up to about 30 feet and required in considerable numbers, should be pre-stressed on one of the long-line systems where the concrete is cast around pre-tensioned wires; but for longer lengths and where the arrangements for pre-tensioning are not available, beams should be post-tensioned as in the Freyssinet and Lee-McCall systems, the relatively high cost of the cones and end anchorages being less noticeable in the longer beams.

Steel for Girderwork

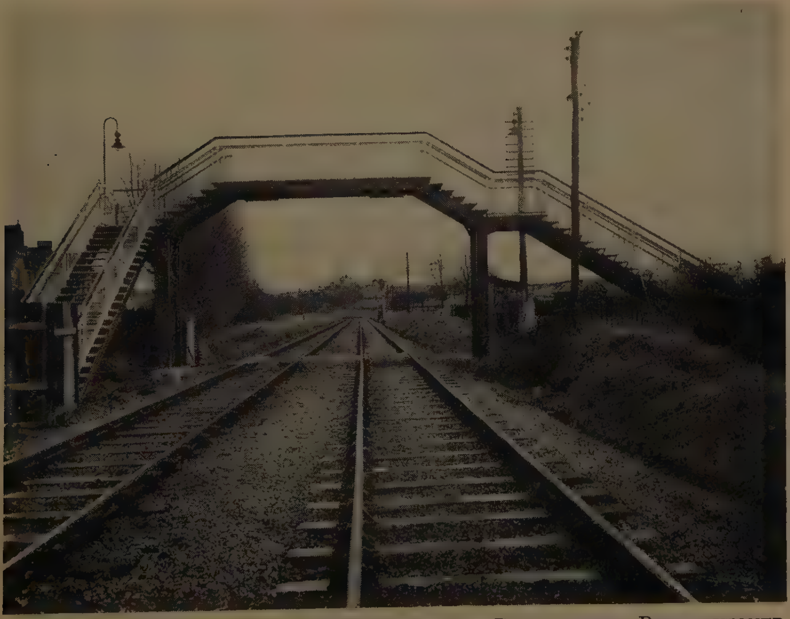
Because of the relatively lower initial cost, the flexibility of design and comparatively short periods of occupation required for erection during

Fig. 13



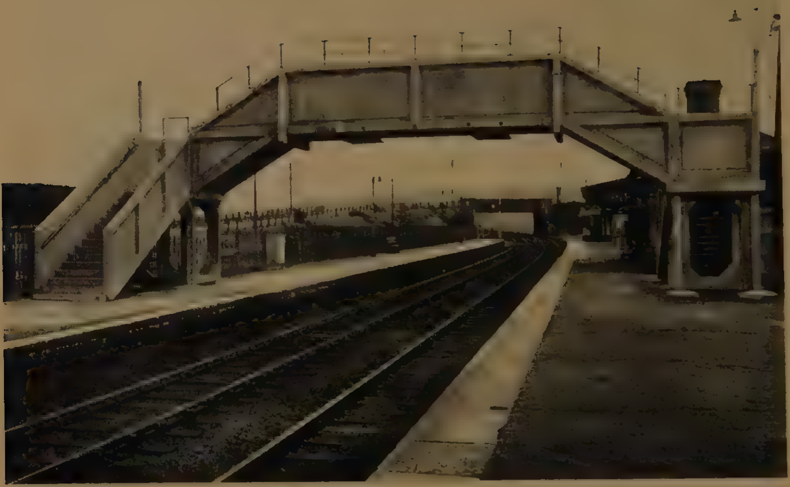
Road Bridge of Pre-cast Pre-stressed Concrete Beams made on a Long-Line Pre-tensioning System

Fig. 14



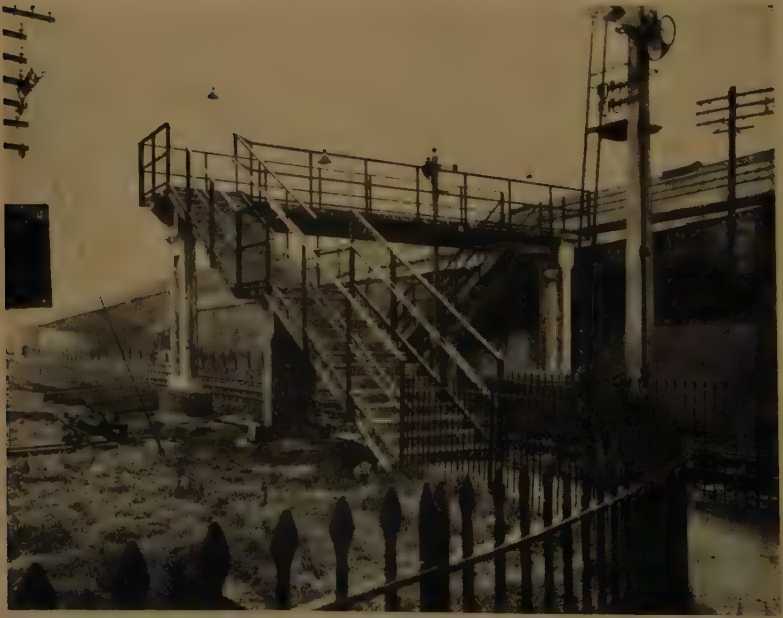
Welded Rigid Steel-Frame Footbridge with Pre-stressed Pre-tensioned Concrete Planks

Fig. 15



Pre-cast Reinforced-Concrete Footbridge

Fig. 16



Tee-Beam Pre-cast Reinforced-Concrete Footbridge reconstruction, mild steel is the most commonly used material for underline bridges. For spans up to 20 feet in length, it is generally economical to use rolled-steel beams for the main girders; for longer spans up to 80 feet, plate girders are of riveted or welded mild steel; and for still longer spans up to about 110 feet (the largest plate girder that can be conveniently transported in one piece by rail) the stress-carrying web and flanges of riveted plate girders are of high-tensile (low-alloy) steel having an ultimate tensile strength of 37–43 tons per square inch and a yield stress of 23 tons per square inch. The use of high-tensile steel is economical when full advantage can be taken of the higher permissible working stresses (12·5 tons per square inch in tension, compared with 9·0 tons per square inch for mild steel) and when the extra cost including the shop work is not more than 38 per cent greater than with mild steel; but in the shorter spans, the minimum thicknesses of the plates and sections dictated by practical reasons prevent the higher permissible stresses being realized in the design. High-tensile steel is harder to work than mild steel, but since the areas, thicknesses, and number of holes are reduced, it may in general be said that the net fabrication cost per ton of riveted high-tensile steel increases proportionately to the saving in weight. Taking other items of cost into account it may be concluded that, if the gross cost of plain material be taken as unity and the percentage saving in weight resulting from the use of high-tensile steel as p, then for a riveted plate-girder, erected, there will be a total saving in cost[2] equal to about $(\frac{1}{2}p - 3)$ per cent.

A considerable saving in weight can be effected by electric welding, and welded mild steel compares favourably with riveted high-tensile steel; but, for railway bridgework, the welding of high-tensile steel is not yet considered satisfactory. Moreover, for plates thicker than ½ inch and for all rolled sections "weldable H.T.S." to B.S. 968 has yield and ultimate stresses about 10 per cent lower than those of high-tensile steel to B.S. 548, thus reducing the possible saving in weight due to welding.

The liability of steel to corrode is its only disadvantage; but provided that the design gives easy access to all parts, and the specification includes proper attention to surface treatment involving complete removal of mill scale and the application of the priming coats of paint on a clean surface, maintenance costs can be kept down to a minimum. Periodical painting should be the only maintenance requirement, and when in the past there have been other troubles, they could, almost invariably, be attributed to unsatisfactory design. A lack of proper appreciation of the allowances to be made for certain loading and stresses—in particular, lateral, traction, and braking forces, and secondary and deformation stresses caused by temperature changes, floor interaction, and so on—has left a legacy of costly and often recurring repairs, which it is hoped will be avoided by better design in the future.

Aluminium Alloys

Present-day prices show that no economy is to be expected from the use of aluminium alloy instead of steel for girderwork, in spite of the expectation of life of the former being double that of the latter. But there may be instances where the saving in dead weight in long spans makes the aluminium-alloy girderbridge attractive by virtue of the saving that may be expected in erection costs. Such a case[3] is the single-track Grasse River bridge in the U.S.A. where a 98-foot long aluminium girder deck-type span weighing 24 tons was lifted into place in one piece, whereas an adjoining and similar span built in steel and weighing 57 tons, was too heavy to handle in one piece, and the main girders had to be erected separately and braced together afterwards.

FABRICATION

Steel Girderwork

Generally, shop-labours should be kept to the minimum, and any curved work will increase the cost. In riveted construction in particular, the cost will be kept down if the following processes are avoided as far as possible :—

(1) Smith work.
(2) Machining.
(3) Gas-cutting.
(4) Making re-entrant cuts, impossible to shear.

The fullest use should be made of batch-drilling ; and designs in which there is the minimum of templet-making will show economy.

Single-rivet connexions, such as are sometimes seen in latticing, are always troublesome and lead to poor-quality riveting. Some designers claim that the use of the same diameter of rivet throughout a plate girder shows a saving even though smaller rivets could be used in the stiffeners than are used in the flanges. Countersunk rivets are not generally so satisfactory for structural work as snap-headed ones ; apart from the extra loss of section in tension members, there is greater possibility of defective workmanship. The depth of countersink, both for heads and points, is often made approximately seven-sixteenths the diameter of the rivet ; this is too shallow for bridgework because such rivets are likely to become loose under vibratory loads. Where countersunk rivets—particularly flush countersunk—are unavoidable, the countersinking should be carried well down into the material as in Table 8 and 9 somewhat in line with Lloyd's and Admiralty practice and with the 1941 addendum to B.S. 275.

Welded construction, apart from saving up to 20 per cent in weight of steelwork, can greatly reduce shop-labours. Girders should be designed for assembly in jigs and arranged, so far as possible, for welding by automatic processes. The reduction in size of single flange-plates ceases to be worth while when the cost of the butt-weld exceeds the value of the metal saved. Intermittent fillet-welds, complicating automatic welding and

TABLE 8.—RIVETS, COUNTERSUNK HEADS

Diameter of rivet : inch	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$
Depth of head : inch	$\frac{3}{8}$ or $\frac{13}{32}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$
Angle of countersink : degrees	60	45	45	45

TABLE 9.—RIVETS, COUNTERSUNK POINTS

Thickness of plate : inch	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$
Depth of countersink : inch	$\frac{11}{32}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{3}{4}$
Angle of countersink : degrees	60	60	45	45	45	45	45

often exposing joints to corrosion, should be avoided. Flanging is often cheaper than welding, but, for cold bending, the steel must have a lower tensile breaking strength, as provided in B.S. 153 but not in the Code of Practice for Simply Supported Steel Bridges or in B.S. 15. In truss-girders some saving is sometimes effected in all-welded web members; but it is more economical to rivet the chords and the main joints, the welding of site joints being undesirable in any case. Bearings, including knuckle bearings, previously made in cast steel, can be made very much more cheaply with thick slabs of mild steel and welding.

Shop Erection

Trial assembly at the makers' works is essential if time and money are to be saved when the bridge is erected at site. In the case of an order for a number of exactly similar spans where steel-bushed jigs have been used to ensure accuracy in cutting to length and the drilling of holes, it is necessary to assemble only one span at the works; but for this the parts will be chosen at random.

BRIDGE ERECTION AND MAINTENANCE

Planning for Site Erection

Site work invariably costs more than shop work. Work is interrupted by the weather; and when the erectors have to wait for occupations or for the passage of trains, the costs rise steeply. Obviously therefore, the design should involve the minimum of field-work. The bridge should be so designed that all parts can be assembled without double handling.

It should be possible to swing all cross-girders into position without "crooked" slinging or having to move the main girders laterally; and similarly stringers or rail bearers should be designed to swing in between the cross-girders without the need for disturbing the latter to complete the joints.

Pre-assembled parts should be as large as can be conveniently handled by the available crane power, and all site joints must be readily accessible for riveting or bolting. Indeed, considerable economies are claimed by the use of high-strength-steel bolts instead of rivets. These bolts, tightened to a predetermined torque and not tightly fitting in the holes, give, because of the clamping force, a stronger fastening than the hot-driven rivet; and experiments show that the high-strength bolt will stay tight under exacting conditions of extreme vibration in which hot-driven rivets have worked loose.[4] The welding of site joints is not desirable because conditions are unlikely to be favourable to the operator and, especially where work has to be completed during limited occupations, such work is apt to be of poor quality and undependable.

Painting under Routine Maintenance

All parts of the steelwork must be readily accessible to the brush. Deep stepped bearings, bedstones well chamfered back, and the elimination of end plates on plate girders go a long way towards improving maintenance facilities at the ends of spans (*Figs 17* and *18*). If pilasters are required for aesthetic reasons, there must be plenty of room for the painter to get at the girders behind them. Travelling cradles should be a permanent feature under all large spans difficult of access. Fixed steel ladders should be provided to give access to all "working" bearings, etc., which require frequent inspection.

CONCLUDING REMARKS

The Influence of Cost on Future Designs

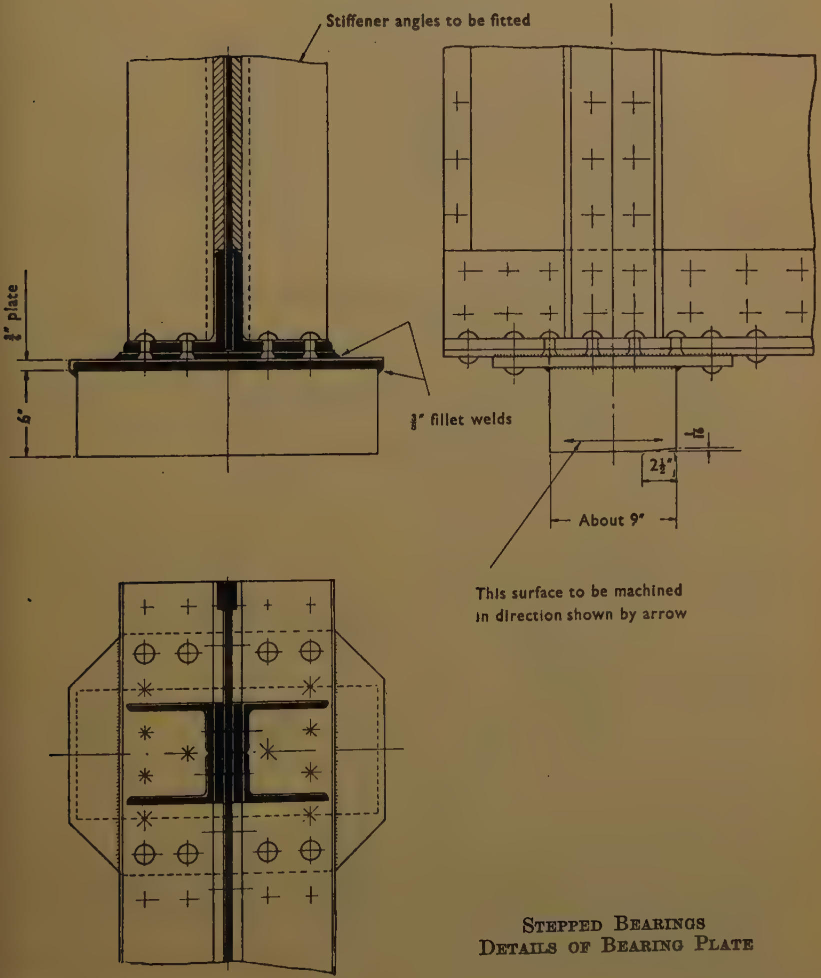
Most bridge engineers have their own ideas about details of design and in the past there has been much evidence of conservatism. The Author believes, however, there will be some agreement on the general principles expressed in this Note. Shortages of material and the rising cost of labour, particularly in the field, will continue to influence design; it is important to ensure that the trend in the future will be towards reduced cost of erection and maintenance.

ACKNOWLEDGEMENTS

The Author is grateful to Mr A. S. Quartermaine, C.B.E., M.C., B.Sc., Vice-President I.C.E., Chief Engineer, Western Region, British Railways, for permission to present this Note and he wishes to express his thanks

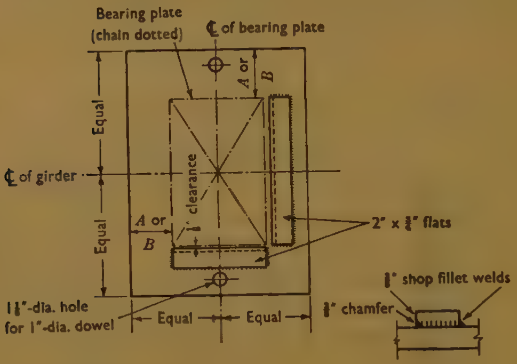
to members of his staff for their help, and in particular to his assistants, Mr P. G. Payne and Mr F. M. Easton, A.M.I.C.E.

Figs 17


Stiffener angles to be fitted
¾" plate
6"
⅜" fillet welds
2½"
About 9"
This surface to be machined in direction shown by arrow

STEPPED BEARINGS
DETAILS OF BEARING PLATE

Figs 18



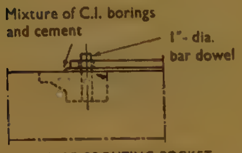
Minimum thickness of bed-plate: 1"

Working pressure between bed-plate and bed-stone to be as in B.S. 153, Pt.3, Cl. 28
Thickness T of bed-plate to be proportioned on the following formula:

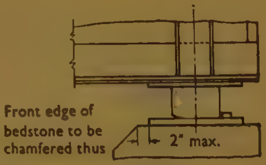
$$T = \sqrt{\frac{3W}{fb}\left[A^2 - \tfrac{1}{4}B^2\right]}$$

where:-

A denotes greater projection of plate beyond bearing plate in inches
B " lesser " " " " " " " "
W " load on bed-plate in tons per square inch
fb " permissible bending stress (9 tons per square inch)



DETAIL OF GROUTING POCKET
IN BED-STONE



Stepped Bearings. Design of Bedplates

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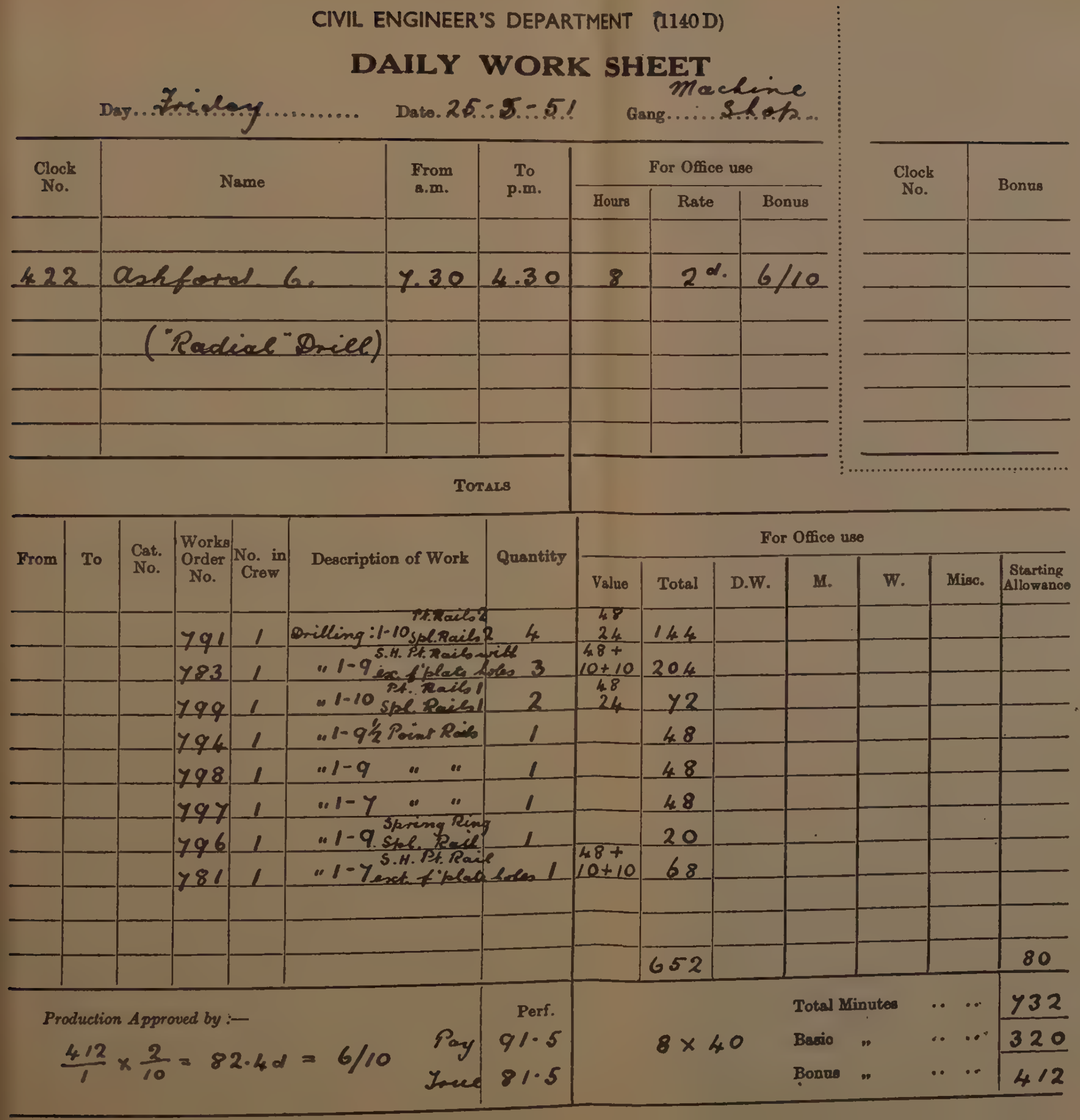
1. Arthur Dean, "Pre-stressed Concrete applied to the Construction of Railway Bridges and Other Works," Railway Paper No. 44, J. Instn Civ. Engrs, 1951.
2. O. A. Kerensky, "Use of High Tensile (Low Alloy) Steels in Bridges," Pubs. Int. Assoc. Br. Struct. Engng.
3. Ctee Rep., "Iron and Steel Structures," Amer. Rly Engng Assn, vol 51, No. 485, p. 441 (Jan. 1950).
4. Ctee Rep., "Impact and Bridge Stresses," Amer. Rly Engng Assn, vol. 51, No. 484, p. 287 (Dec. 1949).

APPENDIX I



CIVIL ENGINEER'S DEPARTMENT (1140 D)

DAILY WORK SHEET

Day *Friday* Date *25-5-51* Gang *Machine Shop*

Clock No.	Name	From a.m.	To p.m.	For Office use			Clock No.	Bonus
				Hours	Rate	Bonus		
422	Ashford G.	7.30	4.30	8	2d.	6/10		
	("Radial" Drill)							
	TOTALS							

From	To	Cat. No.	Works Order No.	No. in Crew	Description of Work	Quantity	For Office use						
							Value	Total	D.W.	M.	W.	Misc.	Starting Allowance
			791	1	Drilling: 1-10 Pt. Rails 2 Spl. Rails 2	4	48 24	144					
			783	1	" 1-9 S.H. Pt. Rails with ex. f'plate holes	3	48 + 10+10	204					
			799	1	" 1-10 Pt. Rails 1 Spl. Rails 1	2	48 24	72					
			794	1	" 1-9½ Point Rails	1		48					
			798	1	" 1-9 " "	1		48					
			797	1	" 1-7 " "	1		48					
			796	1	" 1-9 Spring Ring Spl. Rail	1		20					
			781	1	" 1-7 S.H. Pt. Rail ext. f'plate holes	1	48 + 10+10	68					
								652					80

Production Approved by :—

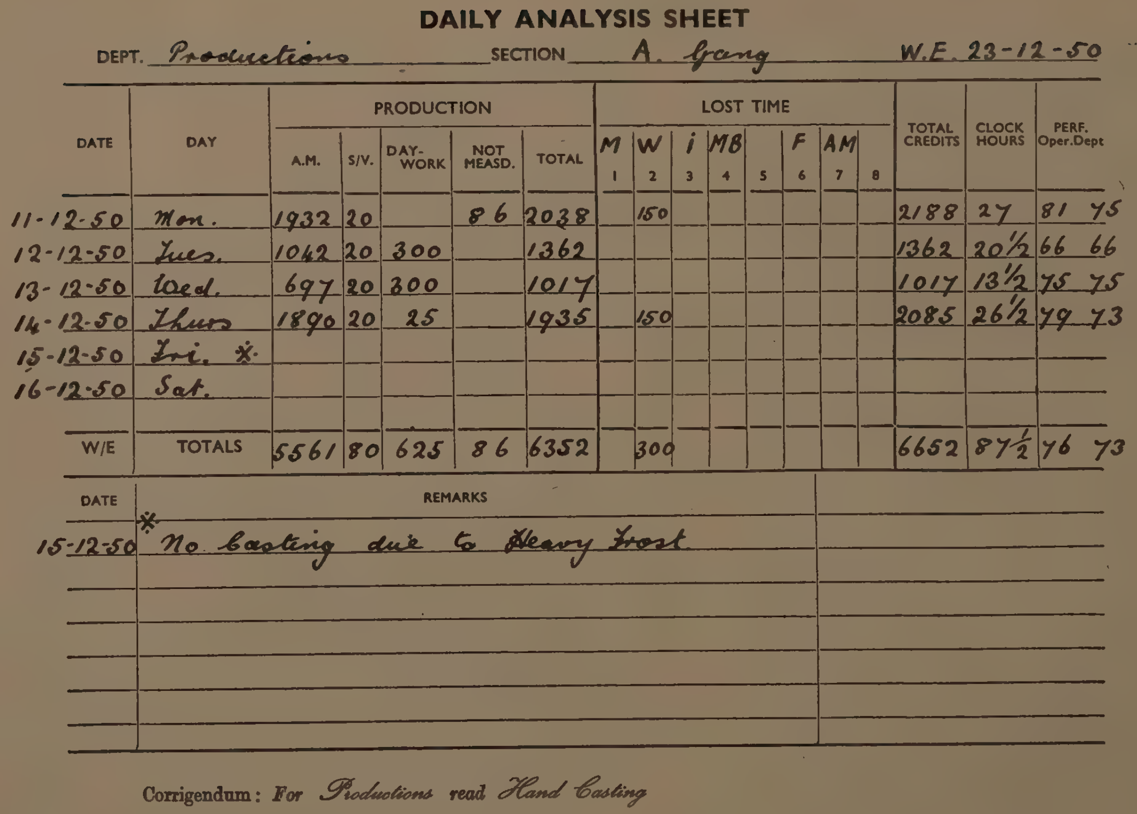
$\frac{412}{1} \times \frac{2}{10} = 82{\cdot}4d = 6/10$

	Perf.
Pay	91·5
Total	81·5

8 × 40

Total Minutes 732
Basic ,, ... 320
Bonus ,, ... 412

APPENDIX II



DAILY ANALYSIS SHEET

DEPT. *Productions* SECTION *A. Gang* *W.E. 23-12-50*

DATE	DAY	PRODUCTION					LOST TIME								TOTAL CREDITS	CLOCK HOURS	PERF. Oper. Dept	
		A.M.	S/V.	DAY-WORK	NOT MEASD.	TOTAL	M 1	W 2	i 3	MB 4	5	F 6	AM 7	8				
11-12-50	Mon.	1932	20		86	2038		150							2188	27	81	75
12-12-50	Tues.	1042	20	300		1362									1362	20½	66	66
13-12-50	Wed.	697	20	300		1017									1017	13½	75	75
14-12-50	Thurs	1890	20	25		1935		150							2085	26½	79	73
15-12-50	Fri. *																	
16-12-50	Sat.																	
W/E	TOTALS	5561	80	625	86	6352		300							6652	87½	76	73

DATE	REMARKS
15-12-50	* No casting due to Heavy Frost

Corrigendum: For *Productions* read *Hand Casting*

WEEKLY ANALYSIS

LABOUR UTILIZATION

WEEK ENDING	CONCRETE WORKS	DIRECT LABOUR	- - -	HAND CASTING					
	HOURS	OPERATING	DAY-WORK		UNMEASURED PRODUCTION		TOTAL	PERFORMANCE	
								OPERATOR	DEPARTMENT
				%		%			
Jan 3	1,077	70,198	5,097				75,295	70	70
,, 10	1,066¼	71,885	6,389				78,274	74	74
,, 17	1,155¼	83,409	4,578				87,987	77	76
,, 24	1,141¼	81,279	3,214				84,493	76	74
,, 31	1,183	78,186	5,100				83,286	72	69

PAY ROLL AND COSTS

WEEK ENDING	PAY ROLL									COST PER 1,000 —PENCE								
	DIRECT			INDIRECT			TOTAL			DIRECT			INDIRECT			TOTAL		
										Actual	Standard	%Excess	Actual	Standard	%Excess	Actual	Standard	%Excess
Jan 3	141	16	11	48	0	6	189	17	5	457	415	10	155	156	1	612	571	7
,, 10	144	9	5	48	19	3	193	8	8	448		8	152		3	600		5
,, 17	158	15	-	52	10	1	211	5	1	437		5	145		7	582		2
,, 24	154	19	9	50	2	6	205	2	3	445		7	144		8	589		3
,, 31	156	11	9	49	9	3	206	1	-	457		10	144		8	601		5

Corrigendum: Under "Labour Utilization," right-hand column, *for* "69" *read* "70"

Discussion

Mr F. E. Campion, referring to Introductory Note No. 1, observed that it might be of some interest to give an account of the effect of the incentive schemes which Mr Stevens had advocated and which had been put into force fairly extensively on the Southern Region. The first scheme had been started about 5 years ago in a concrete depot. It had been very successful, and had now been extended to all the manufacturing and supply depots in the department. The total number of staff involved was upwards of 550, in four different centres.

Before the different schemes had been brought into operation, the performance levels of the staff on the ordinary hourly rates had been ascertained. They had varied from the minimum of 33 Allowable Minutes of work per hour to a maximum of 63. The higher figure was at one of the works where, although the men were more or less hourly rated, there had been a sort of crude bonus scheme in force, the chief effect of which had been to give them a very large addition to the pay packet without any very material increase in output. Since the introduction of the schemes, the average level of performance was now 83 A.M.'s of work per hour, with a minimum of 75 and a maximum of 88. There was a considerable variation, of course, in the improvement, but the general average was about 60 per cent higher than it had been before the schemes were introduced.

From the production-cost point of view, labour costs had been reduced by from 28½ to 30 per cent, whilst the staff earnings were roughly from 30 to 33 per cent higher than the normal hourly rate. So far as increased production was concerned it was difficult to give any comparative figures, (owing to the adoption of new standards for many products) except in one case where there was a very old point-and-crossing shop which, before the introduction of the scheme, had turned out seventeen units of switches and crossings per week. The number of the staff had been reduced by 25 per cent, owing to retirements, but the output had increased from seventeen units to thirty-three with a corresponding decrease in cost.

All the schemes so far introduced on the Southern Region had been confined to depots and works, but there did not seem to be any reason why they should not be extended to most of the work in the Department at some time in the future. It would present a somewhat different problem, inasmuch as permanent way work and general building and structure maintenance were scattered. Mr. Campion suggested, however, that not only was that extension possible and advisable but, in fact, it was imperative if the railways were to solve their present problem of staff shortage and to meet the need for a very heavy reduction in expenditure.

Mr E. Bateson, referring to Mr Berridge's Note, confined his remarks to one or two particular points.

When speaking of economy as applied to bridges, it was necessary to have a very clear idea of what was meant. It did not mean the cheapest in first cost, but the cheapest in total cost throughout the useful life of the bridge. It could safely be said that renewals of railway bridges designed 30 to 40 years ago had in many cases been necessitated by increases in locomotive and train loads, although in some instances deterioration had prevented the strengthening which might otherwise have been possible. The life of a bridge designed to present-day loading standards was, owing to limitations imposed by tunnels, bridge clearances, and so on, more likely to be determined by deterioration than by increases in loading. It followed, therefore, that maintenance costs were a very important factor in the total cost of a bridge, and a design which would require the minimum of maintenance, although costing more in the first place, might be cheaper in the end, whilst at all times it would be in a better condition than a less easily maintained structure.

In that connexion, good surface preparation was important. He used that expression advisedly, rather than " cleaning of steel," because it was necessary to produce a surface which was suitable for taking the protective material adopted. In that respect, metal-spray protection was well worthy of consideration.

Mr Berridge had rightly taken some account of maintenance painting, but would no doubt agree that it was very difficult to assess it at all accurately. Conditions might be entirely different on two different jobs, even when they were situated fairly close together. Painting facilities were therefore an important design factor, and Mr Bateson would suggest that the in-turned type of lower chord shown in *Figs 11* was more difficult to maintain than the chord with the outward flanges. In many cases the vertical members in through spans of that type might be only about 15 inches wide, the bottom chords consisting, probably, of 3½-inch or 4-inch flanged channels and giving very little space in which to get to the inside of the chord for painting. In the case of the chord with the outward-turned flanges the problem of the cross-girder connexions could be overcome without difficulty, whilst the verticals and, to some extent, the diagonals could be taken down inside the chord, thus providing stiffness in the form of diaphragms.

With regard to the use of truss spans with curved or polygonal chords—the members being straight between panel points and not curved—then, provided that the extra labour cost did not exceed the extra material cost, the curved chord design should be adopted. It gave a more aesthetically pleasing bridge, and it economized in steel, which, at the present time, was becoming almost a " precious " metal. In that type of chord, notwithstanding the change in slope, the upper lateral bracing connexions could be simplified by offsetting the oblique butt joint between adjoining sections of chord slightly beyond the intersection point of the chord and web members. The sway struts of the upper lateral system could then be normal members set square with the chords. It would be found on investigation that the amount of eccentric stress introduced by that offset was very small, and it was due only to the increment of stress at that point, and not to the total stress in the chord.

Mr J. R. Dallmeyer observed that supervision was a subject on which it was very difficult to lay down any rules, and therefore one which needed the most careful consideration by the District Engineer himself. The required supervision depended upon its quantity and its quality. The Engineer had control of the former within the limits of the numbers of his staff, but he had not control of the latter, beyond his judgement of the supervisor's capabilities. In other words, he could vary the number of men to be supervised, but he could not alter the capabilities of the supervisor.

In the management of a District, the test of the economy was the cost of the various works compared with similar ones in other Districts, and also the extent to which the annual programme, specially authorized works, and other works were carried to completion, leaving the minimum amount of such work to be carried forward to the following year.

In Mr Dallmeyer's experience as a District Engineer, it was the small District which was most economical in the sense just referred to, and the reason was the closer supervision which could be given by the District Engineer, his Assistant and his two Chief Inspectors, who could obviously exercise their supervision in a greater quantity—not necessarily quality—on fifteen Inspectors than on the thirty or more to be found in a large District. If his experience in that matter were accepted, it meant that the Inspectors required a considerable quantity of supervision from above, and it also meant that, if they were better supervisors, the whole of the railway would be more economically run; and that held good whether his experience was right or not.

How, then, could it be brought about? The qualities required in a good Inspector were technical knowledge, organizing ability, enthusiasm, leadership and personality.

Technical knowledge was necessary so that the work carried out was of the best quality and so that up-to-date methods were used with both materials and labour. It was essential that the supervisor's knowledge in this sphere should be such that his men respected him for it. Permanent Way and Re-laying Inspectors were recruited almost exclusively from lengthmen and re-layers. At no time did they receive any special training other than their voluntary efforts to pass the examinations "A" and "B." They passed through the grades of sub-ganger and ganger, and the resulting inspector was usually a good supervisor; but Mr Dallmeyer was quite convinced that special training at each of the promotional steps would secure something better in the end. Were the raw recruit, on joining the railway, given a short course on simple things such as how to use a shovel, how to use a spanner, how to hit a key—that was almost as difficult as hitting a golf ball—and so on, and were he told the reasons for the many operations of maintenance and renewal, it would lead to a general improvement in his work and, in particular, in starting the future Inspector on the right lines. Advanced instruction should be given to selected men as they progressed upwards. Gangers' meetings were a fertile ground for instruction and training, as well as providing opportunities for personal contact. Works Inspectors were usually drawn from the tradesmen, so that the early training was not so important, but further training in up-to-date methods would ensure economic results.

So far as organizing ability was concerned, certain men were gifted, but it could also be learned. On the Permanent Way side it was learned by experience, and that was amplified now by courses for Inspectors. On the Works side it was greatly simplified by the " Cabinet " system of works control, which Mr. Dallmeyer was glad to know was becoming universal throughout British Railways. Training for selected gangers and for senior tradesmen in organization would give a reward.

Enthusiasm meant keenness to make a particular gang, section, or district outstanding amongst others. Pride in one's work and a desire to please were the main constituents, and it was brought about largely by the interest taken by those in authority ; it quickly died if not recognized. The personal attention of the District Engineer was, in that, most important.

Leadership and personality constituted an attribute which it was most difficult to provide if it were not present originally. It was probably that part of supervision which had led Mr Terris to say that supervisors were born and not made. It was either inherent or learned in early youth. Technical knowledge went some way towards helping to provide it, but there was little training to give it.

Summing up, Mr Dallmeyer's conclusions were that the training given to Inspectors at the now-established schools had given excellent results, but that Regional, District, or Section training should be given to men before they reached the rank of Inspector.

Mr R. L. McIlmoyle observed that all the Authors had taken " economy " in the present context to mean economy in the use of man-power. With the national policy of full employment, an industry which was finding it more and more difficult to make ends meet was being forced to follow what the Americans had done for years, and to make the most economical use possible of the men that they had available.

If, however, the greatest possible economies in the use of labour were to be achieved, it was necessary to increase the amount of supervision. That was a point which had been emphasized in the Introductory Notes, but it had probably not yet received the attention which it really needed if the maximum value were to be obtained from the efforts which everyone was now making.

Mr Terris had mentioned specifically that the small savings which could be made per man did, in total, justify a very considerable increase in the amount of supervision provided, and, unless that supervision was provided there would be a vicious circle, and the economies would not be obtained.

Mr Stevens, when initiating the Discussion, had traced modern incentives back to Taylor, but in fact incentives went very much further back than that. In spinning and weaving, before the Industrial Revolution, there was payment by results, and even further back there was the case of the patriarchs, Laban and Jacob. Incentives, with such a long history behind them, were almost the only possible way, or at any rate the common way, of persuading a man to produce more.

Mr Stevens's Note had drawn attention to four main points. Mr McIlmoyle emphasized the first of these—the question of bonus payments to supervisors. The question was very pertinent, because in some cases men were earning almost twice as much as the supervisor, and there was a lack of interest on the latter's part in seeing that the work was carried out as it should be and that a man had sufficient work to keep him fully employed for his maximum payment. That point required to be followed up at a much higher level.

The statement made in the last paragraph of Mr Stevens's Note, namely, that " Experience has shown that the more difficult the subject is to study and bring into the field of incentive working, the greater is the financial saving to be realized," was too wide a generalization. It was not always necesssarily true that the more difficult the subject the greater the financial saving.

Referring to Mr Taylor Thompson's remark about " traffic sketches," Mr McIlmoyle said that he had often found that a scheme based on a traffic sketch could have been very much improved if the Engineer had been allowed to go into it properly and develop it. Mr Taylor Thompson's point regarding the relationship between organization and management was too often overlooked.

Mr Terris, in his Conclusion, had given three alternatives. Mr McIlmoyle did not regard them as *alternatives* ; any one of the three by itself would not give the answer required ; at least two were necessary—probably all three—to provide a completely integrated and balanced scheme of supervision.

Mr Berridge had confined his Note to economy in the design of bridgework, but had mentioned that economy could be achieved in other directions in design ; it was surprising just how much could be achieved. Mr Berridge had mentioned three factors contributing to economy in design, but there was an additional one which did not arise in the case of bridgework but did in other forms of design ; providing a design which was economical from the user's point of view. That was very important in present circumstances, when economy in labour was being considered, and it arose in schemes for handling goods and others of that type.

Mr Berridge suggested that, on subsidiary lines, open-deck bridges should be adopted. Mr McIlmoyle regarded that as a very retrograde step, and from the point of view of ultimate economy he doubted very much whether it could be shown to effect a saving. In Table 6, where comparative costs were given of superstructures there was a small expenditure shown for painting pre-stressed pre-cast concrete units, but it was difficult to know where paint would be put on such a structure. With regard to footbridges, was a welded bridge more economical than concrete, having regard to ultimate cost? It should be possible to make a concrete bridge which would prove to be ultimately more economical than, and as light and attractive as, a welded steel bridge.

Mr A. M. Sims, who confined his remarks to design economy, observed that it was usual to make certain assumptions with regard to the supporting of girders with the object of simplifying the stress calculations. With the advance in theory, and, particularly, in research, it would seem that the support factors used should agree much more closely with the practical conditions obtaining in the actual work. For example, B.S.153 stated that "cross-girders shall be designed as simply-supported beams spanning between the centres of main girders." In Mr Sims's experience, the cross-girders in most cases did not span between the centres of the main girders, nor were they simply-supported. End-fixing moments could be calculated quite reasonably, and if the relief of stress allowed under one particular code was claimed he thought that that would result in economy of material.

He was glad to see that Mr Berridge had referred to deformation and secondary stresses. It was surprising to find that there was still a tendency to neglect them. They could be calculated, and, if the empirical range at present allowed to cover these in the codes or standard specifications were cancelled, it would be logical to use total working stresses much nearer the elastic limit than at present, and this in itself would result in economy of material.

For some time past a good deal had been heard about the plastic theory, and that theory visualized the use of higher working stresses than had been customary in the past. That seemed to support the case for using higher permissible working stresses with the ordinary elastic theory.

The question of the flanges of the bottom chord being turned inwards was an old bone of contention between Mr Berridge and himself. Mr Sims agreed that a cleaner connexion could be made with the cross-girder, but he thought that that was the only advantage. There was no difficulty in making quite a satisfactory connexion of the cross-girder when the flanges of the bottom chord were turned outwards. There were other disadvantages attaching to the turning of the flanges inwards. As was well known, for theoretical reasons, members should be taken down as closely as possible to the nodal points, particularly for compression members, in order to avoid eccentric loading. A concentrated joint was really very desirable and could be obtained much more easily when the flanges were turned outwards. Turning flanges inwards required larger gussets and, consequently more material. The smaller the gussets the greater the economy, and also the better the agreement with theory. The end-raker could be taken down quite close to the end node when the flanges were turned outwards, and this was done by stopping-off the lower chord and giving preference to the end-raker.

Another way in which he thought that economy could be obtained was by the proper relation of correct and advanced calculations to the code or standard specification which had been adopted in connexion with the design. It seemed to Mr Sims that standard codes and standard specifications should have the effect of standard methods of calculation being required by Chief Engineers in strict conformity with such regulations. Mr Sims had put forward the suggestion in India that if such detailed calculations were published as technical Papers, particularly within the departments to which they referred, then economy might be engendered by the discussion and criticism which they would arouse. Furthermore, there was no better method of training keen and young engineers in design. Not only would those engineers absorb the application of theory, which was often a real difficulty, but such publication would greatly enhance their training in the ways of their own department. Mr Sims felt that engineers in Government service in all countries of the Commonwealth could give a great lead in that respect, particularly with regard to type designs, which should be outstanding examples of true economy.

Theory should be the civil servant of construction. After all, theory was used for a certain number of hours in the production of the design, but the structural work lasted for 60 years or more.

Mr F. Turton said that, apart from small details, he agreed with Mr Berridge's exposition of the broad principles of the design of bridge superstructures. He would, however, shift the emphasis a little here and there, which might or might not appear to alter the picture, depending upon the point of view.

Mr Berridge had rightly pointed out that economy in design included first cost and maintenance. The latter, in Mr Turton's opinion, was of extreme importance; the design, he thought, should be aimed at incurring as little future expenditure as possible. The accountant's figure, quoted by Mr Berridge, of 55 years for the life of steel and reinforced concrete, which presumably included wrought iron also, seemed utterly divorced from reality. Some steel bridges were already of that age, and many of wrought iron were 80 to 90 years old.

Recent inquiries made by Mr Turton revealed that in the 80 years between 1870 and 1950 approximately 25 per cent of the wrought-iron underbridge superstructures on the late L.M.S. system had been completely renewed. Taking into account the much lower rate of reconstruction which obtained to-day, it would be more than 300 years before the last wrought-iron underbridge disappeared. Even if the present rate of reconstruction were doubled, the last wrought-iron underbridge would remain in being for a very long time.

That meant that emphasis should be laid on preserving existing structures but, when reconstruction did become necessary a structure should be built which demanded the absolute minimum of attention. Viewed in that light, the initial cost was of secondary importance. The bridges which had given the least trouble in the past had been those of brick and masonry construction. It seemed, therefore, that a material which had approximately the same lasting properties as those two was probably the best material to serve for the future; the answer was, he thought, first-quality concrete.

There was a tendency in reconstructions to consider only beam-type superstructures, but there were some cases where arch-types of pre-cast concrete could be used. He would suggest that the first approach to the problem of a new bridge or of a reconstruction should be an investigation as to whether concrete could be used in any form; if not wholly, could it be used in combination with steel and not merely as clothing? Only after having given full consideration to that would Mr. Turton turn to steel as the predominantly structural material. The economics of that were confirmed by Mr Berridge's comparative cost diagrams. What should receive first consideration were the cross-hatched parts at the right-hand end of the diagram in Table 5. Types 3 and 6 were similar in this respect. Type 6 used slightly less steel and needed less construction depth and was easier to erect; this was a type which, with slight modifications, was being tried on the L.M.R., and it seemed to hold great promise. In Table 6 using a similar approach, type 4 would be chosen, and in Table 7, type 2. In short- and medium-span underbridges, therefore, Mr Berridge had shown that reinforced concrete, pre-stressed concrete, and welded steelwork in combination with pre-stressed concrete were the most economical types, not only in maintenance but also in first cost. Mr Turton would go further and, wherever possible, would use pre-stressed concrete, because he believed that concrete in compression was the right way to use it, and there was little chance of it needing any attention at all.

If, however, steel was the only material which would satisfy a given set of conditions, let it be welded. That required very much less attention than riveted steelwork.

He thought that similar principles should apply to spans longer than 40 feet, and also to overbridges. Incidentally, he saw no reason why pre-stressed concrete bridges should not be rolled in.

There were several points of detail in Mr Berridge's Note with which he could not agree. Mr Berridge had shown the use of mesh panels for parapets, but they were extremely difficult to maintain. He also showed the use of steel handrail standards and tubing, even on an otherwise wholly concrete bridge. It was difficult to see the reason for having them, with the necessity to paint them, when the same purpose could be served by the use of pre-cast concrete. In *Figs 10* it was shown that certain details had to be site-welded, which was in contradiction with Mr Berridge's statement that site-welding should be avoided like the plague. Mr Turton agreed with Mr Berridge's remarks about high-strength bolts, but there was one unfortunate omission, in that something might have been said about the drainage of bridge superstructures, which was an extremely important point.

Mr C. B. Trye observed that comments had been made about estimating for new works, and he entirely agreed with the suggestion that a preliminary and very rough estimate should be made. So far as final estimates were concerned, however, he found, on examining a great many where his department had carried out the works, that the estimate in nearly every case bore almost no relation to the actual cost. He had examined seventy-seven estimates, and seventy-two of them had been very wide of the mark indeed, all the estimates being low.

He thought that there was scope for training technical staff, and to a lesser degree artisan staff, in estimating. He did not think that the average "indoor" man, who had not been out on work, quite realized the difficulties that existed on the site. In cricket a good captain would go out and examine the wicket, but rarely did the estimator go out and examine his wicket, because if he did he would find it "sticky" in many places. There were people who would estimate a sort of basic price for excavation without taking into account whether it was loose muck or rock that was to be excavated. In other cases not the slightest notice was taken of whether the men had to travel to their work or whether they had to lodge away from home or whether the work had to be done on Sunday. He would advise anyone who was dealing with an estimate, and who wanted to get it somewhere near the mark, to study the site and the local conditions, and in particular the nature of the ground and the volume of traffic, and not to trust old plans.

Mr Trye had been dealing with a somewhat special job for the last 6 years, in that he had been carrying out more than a hundred jobs by direct labour. Whether that was economical or not was a matter of opinion, and the views of others on that subject would be interesting. His own opinion was that every job must be treated on its merits. Broadly speaking, he thought that direct labour was a success in dealing with artisan jobs but a failure in dealing with permanent way jobs, the reason being that men taken on for direct labour were, for the most part, recruited from the Labour Exchange; they did not realize that they, through their gangers, were responsible for the safety of the public. There was a great difference between digging a ditch in an open field and digging a trench alongside the track. He thought, therefore, that direct labour had very serious limitations so far as permanent way was concerned, but for artisans' work—reconstructing an engine shed, building or repairing a bridge, or building a platelayers' cabin, for instance—it was a very great success.

It had, however, to be carefully watched. Mr Trye thought that the immediate supervision of direct labour was weak. It was not advisable to engage men and then, after 4 or 5 months, to decide who was the best man and perhaps appoint him foreman, because that man would have no railway knowledge. It would be advisable for the direct-labour supervisors to be more or less equivalent to a District Permanent Way Inspector or District Engineer's foreman. If direct labour were concentrated on those lines, Mr Trye thought that it would probably be very successful for artisan work.

Mr E. J. M. Matheson said that he could not add any new figures and facts to the subject of bonus incentive schemes, but he had had experience of task work in the Army as opposed to day work, and the results in favour of task work had been very striking indeed. As a result of that, he had approached the subject of bonus incentive schemes sympathetically, but when trying to apply them to a typical District Engineer's workshop difficulties were immediately encountered. First, the scheme was most suitable for repetition work and there was very little repetition work on the railways. The solution might be to concentrate certain items on a Regional basis in definite District Engineers' workshops. For instance, one District Engineer might specialize in gate-posts for the whole of the Region, and another in tenterhook boards. That was done to a certain extent to-day.

Secondly, the material for even the simplest job had to pass through so many hands. Taking a gate as an example, first of all a gang had to select the timber from the pile outside, the sawyer then had to saw it up to the correct sizes, and lastly the joiners make it into the gate. If the men who selected the timber did not select the best for the job, or if the sawyer did not cut to quite the right size, the final operation would take much longer. It would be difficult, therefore, to assess correctly who was entitled to the rewards.

Thirdly, it was important that every man should have an opportunity to participate in the bonus scheme, but in normal conditions the most skilled man would be put on to the most difficult work, and nine times out of ten the most difficult work would not be repetition work and therefore would not be suitable for bonus; the most skilled man, therefore, might not gain as a result.

There was possibly far more scope for bonus schemes on the permanent way, especially in re-laying, because there the chief variables were the weather and the Traffic Department, and the job was very largely repetition work of a simple nature. Crossing welding might also be a job well adapted to that type of scheme. With crossing welding and re-laying, however, it was the welder and re-layer who were going to get the extra money, whilst one of the men who should get it was the lengthman. It would be interesting to know whether there was any means of assessing the lengthman's work so that he could participate in bonus schemes.

An advantage of bonus schemes which had not been emphasized was that they would make management far more efficient, because if management fell down on the job there would soon be discontent among the men. It was very like the position which arose with the ordinary goods-handling plant in a goods warehouse, which, by virtue of its existence, drove the goods porters to keep pace.

An industrialist had recently told Mr Matheson that a bonus scheme was universal in his factory. When first started it had been believed that there were some jobs which could not possibly be adapted to such a scheme, but it had now been found that almost all types of work could be so adapted.

Mr Matheson felt that it was necessary to go ahead with the investigation of this matter without delay, for two reasons. The first was that the nation was at present passing through a period of full employment; on that account there was very little incentive, and some must be provided. Secondly, if the scheme were introduced and it involved redundancy, owing to the existence of full employment men who were displaced could find jobs elsewhere, and there was no need for anyone to feel that he was depriving his fellow-men of their means of livelihood. Having regard to what Mr Stevens had said, there was also the fact that if it were introduced now the same evil associations would not be linked with Mr Stevens as had been linked with the name of Mr Bedaux.

With regard to the "Cabinet" scheme, Mr Matheson thought it was essential where there was a Chief Works Inspector's organization from the Chief Works Inspector's point of view, quite apart from management. Where there was not the Chief Works Inspector's organization, but a number of Inspectors were responsible for definite areas, it was not quite so important, if the inspectors were competent. The whole subject of Chief Inspectors was a very controversial one. Mr Matheson had been in only one District where the system of having Chief Inspectors operated, and he did not feel that he was justified, until he had been in other Districts, in forming a definite opinion; but he wondered whether Mr Terris was right in saying that having a Re-laying Inspector resulted in a better standard of work. It was doubtful whether in all cases that would be so, because, after all, the Re-laying Inspector was a bird of passage who came to a job and went away, whereas the Section Inspector, if in charge of the work, would do the job well because he had to live with it until he retired or went elsewhere.

⁂ **Mr E. H. Puckering** referring to the statement on p. 108 that "The day-to-day repairs involved in the maintenance of structures call for careful organization if wasteful use of labour is to be avoided" thought that there was a tendency in some cases to order replacement materials without considering whether a saving of material and labour could be made by using an alternative material. That could apply to some classes of fencing, roofing, and flooring materials. A particular case was stair

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⁂ This contribution was submitted in writing.—SEC. I.C.E.

treads. The concourse of one of London's main stations was approached by various flights of steps. Some of them were stone, some had inlaid metal treads, and others had 6-inch-by-6-inch bull-nose clay tile treads, inlaid in cement. No doubt one of the reasons for the last was that if exceptional wear occurred at certain points odd tiles could be replaced, but he doubted whether that was real economy, for it was certainly not efficient.

One of those flights of stairs was 15 feet wide, with a central handrail. It was used by thousands of people during the rush hours. Whenever a tile wore through or was kicked out—and that seemed to happen rather frequently—half the width was cordoned off for 24 hours, causing congestion and inconvenience to the public—to say nothing of the risk of an accident occurring before the steps were put out of service.

In cases of that kind, the substitution of metal strips of the "Ferulan" type for the tiles would be true economy, for they would give years of service without any attention.

Mr G. C. Stevens, in reply, observed that Mr McIlmoyle had referred to the problem of bonus payment to supervisors and apparently felt that some additional payments should be made to them when incentives are being paid to the men they supervised. Mr Stevens had been associated with the introduction and control of bonus schemes for more than 5 years and that was quite the most difficult problem he had encountered. The very nature of bonus schemes, works control, and labour utilization made the Supervisor's task more onerous. The normal position in the workshop had become reversed; no longer did the Supervisors spend much time ensuring that each man did a fair day's work: men now approached their Supervisor with requests to be given sufficient work to ensure a good bonus payment at the end of each day. Several suggestions relating to Supervisors had been put forward and it was hoped that in the near future those concerned would agree to some reward to them.

Mr Matheson had raised several interesting points, but would have to accept Mr Stevens's assurance that as far as workshop staffs were concerned most of the problems, even on the payment of incentives for non-competitive work, had been solved. Taking as an example a non-standard lay-out in flat-bottom track, Mr Stevens mentioned how, at one of the workshops on his Region every component part was manufactured except rails, bolts, and one or two odd signal fittings. The unloading of the pig iron and coke, the moulding and casting, the fettling and assembly of the finished chair in the lay-out, the machining, pressing, and drilling of the rails were all operations which could be, and indeed were, covered by a measured A.M. value and each man, wherever possible, was paid as an individual. The fact that each man was so paid had not been made very clear in the Note, but it was always the aim to pay men in that fashion and not to pay them on a group-shared bonus system.

Mr Stevens entirely agreed with Mr Matheson that incentive schemes were a means of making management efficient. The method described his Note provided Management with information of the individual output of practically every man in the workshop at the end of each week and the cost of producing each thousand units of work compared with the basic target cost. Efficiency of management was measurable almost to the same extent as that of the workmen.

Mr J. Taylor Thompson, in reply, referred to Mr Trye's remark that the cost of carrying out works was usually very different from the estimate. Taking odd jobs, that seemed to be true, but taking a large number of jobs Mr Taylor Thompson had been surprised to find how close the estimates were. In one year, taking the summation of all the specially authorized jobs for the year, the estimates had been within 1 per cent of the actual costs. It might almost seem that the figures had been faked. The individual jobs showed wide variations, but by adding a large number of jobs together the estimates had averaged out.

Mr Matheson had made some comments on the system of having a Chief Works Inspector. Mr Taylor Thompson had had no experience of what he thought must be the Western Region practice, but he had the impression that to have one person in charge of works, who could take control of progress and the distribution of staff, had much to be said for it as compared with the possibly conflicting aims of different and equally important Inspectors who had a staff on which they could call with equal rights. On the subject of Re-laying Inspectors he held strong views. He believed that the Section Inspector should have his attention concentrated on what really mattered, namely maintenance. The standard of the railways depended on maintenance, not on renewal, and it was important to remember that renewal affected only one-twentieth of the mileage annually, whereas maintenance affected it all. The local Inspector should devote his whole attention to maintenance, and therefore renewal should be specialized work.

Mr A. K. Terris, in reply, referred to Mr Dallmeyer's statement that a District, to be efficient, should complete its programme, and suggested that a programme was essentially intended to be completed and that therefore there was something wrong with the organization within the district if that was not achieved in normal circumstances. At the present time, there were difficulties with regard to materials and labour and other factors, so that there might be some excuse; but, if a programme was worth the work put into it, and if the District was properly organized, there should be no question of the programme for the year not being completed.

Mr Terris did not think Mr Dallmeyer was altogether right in saying that the costs in one District should be compared with those in another as a measuring rod of efficiency. It was obvious that Districts differed very much in their characteristics, and the only comparison which could usefully be made was between the costs for one year and the costs for the previous year in the same District, when similar conditions no doubt applied, and allowances were made for certain cost factors. Mr Dallmeyer suggested that a greater proportion of the functions of management could be brought to bear in a small District. Mr Terris was inclined to agree with that, but as one District differed from another so did the amount of top-level management and administration vary, and therefore the problem should, within limits, be approximately the same in all Districts. Many different factors went to the determination of the shape and size of a District—the geographical lay-out of lines and property, the position of various towns where labour could be obtained to carry out the work, and so on.

Mr McIlmoyle seemed to imply that Mr Terris had stated that economy could be obtained only by increased supervision. He hoped he had made it clear that in his opinion the most economical work would be obtained only if the supervision element was adequate. The alternative suggestions put forward on p. 123 had really been intended to promote discussion. There could be no doubt that at the present time, with the difficulties prevailing in regard to labour and the need for bringing contractors on to the railways, the normal quantity and character of supervision done by the primary Inspectors had been completely upset. It followed, therefore, that if jobs were done in a different way and men were brought in who were not knowledgeable about the railways, a greater degree of supervision was required both at the site and higher up. The next alternative was "To increase supervision by the introduction of qualified technical staff at an intermediate level in the District organization." If it were admitted that the quality of the intake of men was deteriorating, and that ultimately this would react on the quality of supervisors, it would be necessary to make up for this factor in some way. A good deal had been said about the value of training, but would that equalize the lowering of the quality of the men who were coming in, and who would ultimately become supervisors on the railway? There was no doubt that, without incentive schemes on the general work of the railway, the Inspectors were having a much more onerous job than they had in former days when "hire and fire" methods were more prevalent and the penalties were much more severe. He had also said that if an incentive bonus scheme were introduced on the permanent way, one could expect less necessity for site supervision of the quantity of work performed by the Inspectors, because it stood to reason that if bonus systems did increase the output, the need for supervision would be lessened. Unfortunately if there were no longer the same need for Inspectors under this heading, there would be other personnel required who would take their place and therefore raise the cost of the overheads on the work.

Mr Trye had dealt with direct labour and seemed to suggest that, for permanent way work, direct labour was a failure, because the men did not appreciate their responsibility for safety. All that Mr Terris could say about that was that on the Eastern Region, where for the past 3 or 4 years there had been about 500 contractors' men engaged on a variety of permanent way works, whilst apprehending the danger, no real difficulty had been encountered nor had this comparable system been found unsatisfactory. Whether it was economical or not was another question, but, properly supervised by superimposing a capable railway supervisor over these men, it could not be said that it had not been successful from the execution and safety angles.

Mr Matheson's point about the Re-laying Inspector had been answered, in part at least, by Mr Taylor Thompson. Mr Matheson took exception to the statement that the use of the Re-laying Inspector would give a better standard of work. There were other arguments which could be put forward for the type of organization which used a Re-laying Inspector. If, however, the Re-laying Inspector was chosen because of his abilities, and was entirely engaged on re-laying work week after week in all its forms, including the use of the prefabricated method, which must in time largely supplant the more manual methods, he did not see how one could escape the fact that the work must be in the hands of re-laying inspectors who, by virtue of being specialists should, other things being equal, produce better results.

Mr P. S. A. Berridge, in reply, observed that Mr Bateson and Mr Sims had risen to his bait regarding the question of whether the flanges of the bottom chords of triangulated girders of through-type bridges should be turned inwards or outwards. That was a subject in which he was very interested and he proposed to deal with it at greater length later in his reply. He did, however, point out that the first bridge in which he had found the flanges turned inwards was the Alexandra Bridge—carrying the single track of the North Western Railway (India) over the Chenab. That bridge had been designed in Mr Bateson's office. The space between the flanges of the 17-inch-by-4-inch channels forming the chords was only 5½ inches but that had proved to be ample for riveting and painting.

Mr Bateson had suggested that the renewals of railway bridges designed 40 years ago had in many cases been necessitated by increases in locomotive and train loads. Mr Berridge agreed with that statement so far as railway bridges in India were concerned. There, stresses in excess of the permissible limits specified in the I.S.R. Steel Bridge Code, which were practically the same as those given in B.S. 153, were not tolerated and bridges had been reconstructed when a heavier loading was introduced. But on British Railways it was a rather different story and the majority of reconstructions and strengthening had had to be carried out because of the deterioration (by corrosion) of girderwork and not because of increases in the loading. In Great Britain, higher stresses and reduced factors of safety had had to be accepted through force of circumstances rather than by choice.

Mr Bateson had mentioned surface treatment and metal-spray protection. Mr Berridge had recently seen a bridge so treated near Merthyr Tydfil. The protection of the steelwork appeared to be very good but it seemed a little early to say how long the protective coating would last, and Mr Berridge did not know how the cost would compare with painting under the usual maintenance procedure.

Mr McIlmoyle had commented on the reference to open-deck bridges. Mr Berridge had included them in his Note because they were so much cheaper than bridges with ballasted decks. He had been concerned with the erection and maintenance of open-deck bridges for 20 years and he had learned to appreciate their advantages. He agreed that the running on and off was not so good as it was on a ballasted track, but he did not appreciate the alleged danger to men walking along an open deck. The gangers were quite used to them in India, and did not complain.

The small expenditure on painting a bridge of pre-cast pre-stressed concrete units given in Table 6 was based on a particular bridge at a town on the coast, where the local authority had asked for a design incorporating tubular handrails. The expenditure covered the painting of those steel tubes.

With regard to the comparative cost of footbridges, the welded rigid frame cost 5 per cent less than the pre-cast reinforced-concrete bridge.

Mr Sims had mentioned the use of the plastic theory and a closer relationship between actual working stresses and theoretical stresses. Mr Berridge thought it was a very good thing that many of the older bridges had been designed on the elastic theory, since it had provided a greater factor of safety than would have been the case had the plastic theory been used. In the past little or no attention had been paid to the stresses in the connexions between the decking and the main girders of through-type girder spans, and had they been designed on the plastic theory, reconstruction would have been necessary long ago.

Mr Turton had said that a life of 55 years for many girder bridges was nonsense. Mr Berridge agreed that many steel and wrought iron bridges had survived for a much longer period, but the figure was that used by the accountants in assessing the value of bridges.

Regarding Mr Turton's reference to rolling-in pre-cast pre-stressed concrete bridges, Mr Berridge agreed that there appeared to be no reason why this should not be done. Indeed a 60-foot span consisting of two pre-cast pre-stressed concrete main girders, with floor units of similar construction, was under design on the Western Region and that bridge would be rolled-in.

With regard to the maintenance of mesh panels in parapets, Mr Berridge had used them on the rigid-frame footbridge, and they had been designed as separate units held in position with bolts so that they could be replaced quite easily.

Returning to the question of the advantages of turning the flanges inwards in the bottom chords of the triangulated girders, Mr Berridge agreed with Mr Sims that web members should be taken down as closely as possible to the nodal points but he pointed out that it was even more important that the axes of all the members should converge on the same point lying on the axis of the chord itself. A study of *Figs 12* showed that when that requirement was met it was not possible to bring the diagonals farther down the chord without loss of symmetry in the arrangement of the rivets connecting the member to the gussets. With regard to the alleged loss of stiffness in the vertical when stopped short as shown in *Figs 11,* that was more than compensated for by the web connexion to the diaphragm in the chord, by the flange connexions through the gussets, and by the cross-girder itself. Whilst agreeing that the gussets should be as small as possible to avoid large deformation stresses in the diagonals, he pointed out that stiffness in the end connexions of a strut did increase its resistance to buckling.

He agreed with Mr Bateson that a girder with a curved or polygonal top chord had a more pleasing appearance than one with a straight chord, but it should be remembered that fabrication and erection were greatly simplified when the joints in the chords were remote from the panel points.

Referring to Mr McIlmoyle's preference for pre-cast reinforced-concrete footbridges, experience had shown that they were not without their maintenance troubles and when spalling did take place the repairs were awkward and seldom satisfactory. In spite of the grouting over erection bolts it was not easy to keep all parts free from rust, and streaks invariably developed and greatly detracted from the appearance of the structure.

The site-welding of the drip plates shown in *Figs 10,* and mentioned by Mr Turton, would be carried out after the bridge had been erected and opened to traffic. Mr Berridge's insistence that site-welding should be avoided referred to stress-carrying welds to be made during erection, the quality of which would undoubtedly be suspect because of the unsatisfactory and hurried conditions in which they would have to be made.

Referring to the comments made by Mr Puckering, Mr Berridge agreed that one of the chief points in designing for economy was to make all parts of a structure subject to rapid wear or deterioration readily accessible for easy and rapid replacement.

ROAD ENGINEERING DIVISION MEETING

4 December, 1951

Brigadier Archibald Cecil Hughes, C.B.E., T.D., B.Sc.(Eng.), M.I.C.E., Chairman of the Division, in the Chair

The following Paper was submitted for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 35

"The Development of Hot-Rolled Gravel Asphalt for Trunk and other Roads in East Anglia" *†

by

Derek John Swift, B.Eng., A.M.I.C.E.

SYNOPSIS

So long as economic conditions remain difficult engineers must, more than ever, turn to inexpensive materials and processes.

The purpose of the Paper is to record how a known, but not fully-proved, process of road-making was applied on a large scale in East Anglia; why it was applied; and what advantages were realized.

The Author stresses particularly the economic advantages which are believed to have been obtained, but which might have remained unexploited if use of the local gravels as aggregate for asphalt had not been dictated by circumstances. This economic advantage is likely to apply equally to future work with the same material when conditions are favourable.

The Paper describes how the work was organized and supervised; it gives details of the specifications adopted, and compares the cost of gravel asphalt with that of other road-making materials laid under similar conditions in East Anglia.

Finally, some suggestions are made which may contribute to the future improvement of the technique of mixing and laying the gravel asphalt.

INTRODUCTION

Road-making practice throughout the whole of Great Britain has undergone remarkable changes during the past decade. These changes have, in general, been resorted to by engineers as a means of meeting:

- the reduction in the numbers of men available for work on the roads;
- the high cost of wages; and
- the need, under present economic conditions, for executing the maximum amount of work for the minimum amount of money.

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† Correspondence on this Paper may be accepted until the 15th April, 1952, and will be published in a later number of the Proceedings, Part II. Contributions should be limited to about 600 words.—Sec. I.C.E.

They have been made possible by the successful development of new road-making plant of many kinds, which has enabled essential work to be done with fewer men than formerly, often at lower cost, and usually in a more satisfactory manner.

In parts of East Anglia the changes have been much more conspicuous than in the remainder of Britain, because, in addition to the changes of method, there has been the widespread use on a substantial scale of a material which had previously been rejected by many highway engineers as unsatisfactory. That material is loosely known as gravel asphalt; it is, in fact, hot-rolled asphalt containing gravel aggregate.

The object of the Paper is to make known what has been done with this material, to describe the lessons which have been learned in the process, and to offer suggestions for its future use and improvement.

DEFINITION

The expression "gravel asphalt" is understood to mean dense hot-rolled asphalt, manufactured and laid in the general manner prescribed in British Standard No. 594,[1] but utilizing gravel as aggregate instead of crushed natural rock or slag. It includes mixtures for both base-course and wearing-course road surfaces manufactured at temperatures of from 250°–350° F., and laid at temperatures of not less than 250° F. The expression also covers a number of variations from the usual specification which have been made with the object of exploiting the cheapest mixes. They are described in detail later in the Paper.

HISTORY

The idea of making use of local gravel as an aggregate for asphalt is not new. It has been put into practice on a relatively small scale in different parts of the British Isles for a number of years, but it is an idea which has not been received with unanimous enthusiasm. The reason is that gravel as an aggregate for coated road-stone has always been regarded with some disfavour, in comparison with the best natural stones and slag. It is more difficult to coat satisfactorily, the manufacturing process requires greater care, and there have been instances of failure caused by the break-down of the adhesion between the tar or bitumen binder and the aggregate, which critics have said would not have occurred if crushed rock or slag had been used instead of the gravel.

The idea of gravel aggregate is most attractive in those parts of the country which are far removed from deposits of rock suitable for road-work and from supplies of slag, but where good local gravel is to be found. These conditions apply particularly to East Anglia (see *Fig. 1*).

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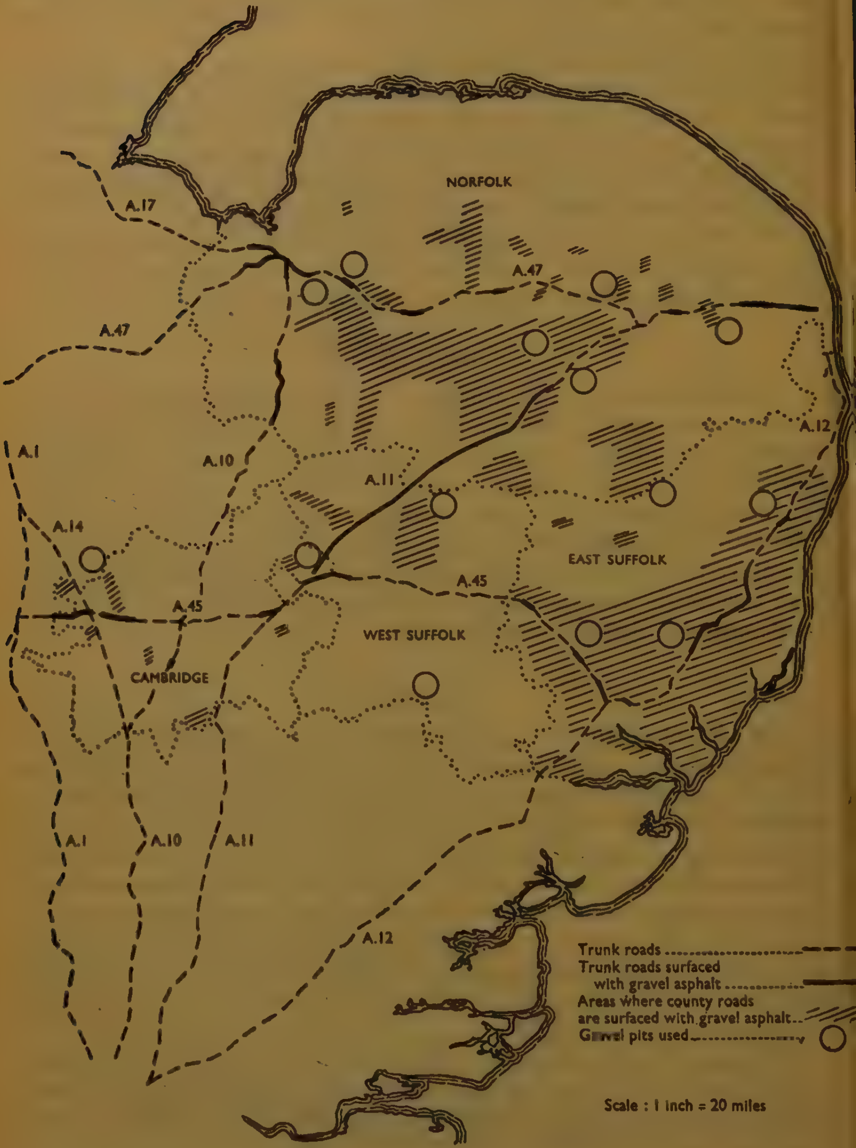
[1] Since the Paper was written, the revised British Standard No. 594 has been published, and it provides for the use of gravel aggregate, as well as for aggregate of igneous, calcareous, or siliceous rock, in the manufacture of hot asphalt.

Fig. 1


NORFOLK
A.17
A.47
A.47
A.1
A.10
A.11
A.12
A.14
A.45
A.45
EAST SUFFOLK
WEST SUFFOLK
CAMBRIDGE
A.1
A.10
A.11
A.12
Trunk roads
Trunk roads surfaced with gravel asphalt
Areas where county roads are surfaced with gravel asphalt
Gravel pits used
Scale : 1 inch = 20 miles

MAP SHOWING LOCALITIES WHERE GRAVEL ASPHALT HAS BEEN MOST USED

Early use of local material for making dense asphalt on a considerable scale in East Anglia appears to have been confined to the County of East Suffolk, where sand-asphalt was first used in 1927. It was then gradually developed by the County Surveyor until, by 1937, it had become firmly established in that county as a reliable and economical road-making material. Much of the early work done in East Suffolk is still in good condition.

There are records of early occasional use in several other counties, but it appears to have been in East Suffolk that the most progressive development took place.

OBJECTS IN USING GRAVEL ASPHALT

Immediately after the 1939–45 war great difficulty was experienced in bringing road-stone into East Anglia from the Midlands because :

- rail haulage was slow and unreliable, owing to the shortage of rolling stock and other disorganization caused by the war ;
- road haulage over long distances was not permissible, since petrol and tires were rationed ;
- shipment by sea was too expensive when the extra handling from the port to the road was taken into account ; and
- the demand for road-stone was, in general, greater than the quarries could meet, suffering as they then were from shortage of labour and other difficulties resulting from the war.

For those reasons, road-reconstruction in East Anglia in 1945 and 1946 had been brought practically to a standstill, although hundreds of miles of roads, which had carried war-time traffic to airfields and tank-training areas, were in a state of collapse. Consequently, it was imperative that an alternative form of construction should be found, making use of the gravel aggregate which occurs locally over most of East Anglia.

Concrete was not considered to be suitable under the circumstances because progress would have been relatively slow and expensive, and more men would have been required than were in fact available.

It therefore became obvious that if a quick solution to the problem were to be found it must be in the form of some mixture of gravel with tar or bitumen. A dense mix with bitumen binder was chosen because it was felt that this would give a material having a long life at the minimum cost in maintenance. Furthermore, it would enable advantage to be taken of the high proportion of sand found in most of the local gravels. It soon became apparent that surfacing material made in that way would be cheaper than either concrete or imported road-stone.

The presence in the area of several firms of contractors owning portable mixing plants and Barber–Greene finishers, which had been employed on airfield construction, was an additional reason for the choice made.

SUMMARY OF WORK DONE

The following is a brief summary of the gravel asphalt laid during the 4 years, ended July 1950, in the counties of Norfolk, East Suffolk, West Suffolk, and Cambridge, where practically all the work has been done :—

Miles of road re-surfaced :

(i) Trunk roads	83·98 miles
(ii) Non-trunk roads	468·15 ,,
Total . . .	552·13 ,,

Tons of base-course material used :

(i) Trunk roads	126,684 tons
(ii) Non-trunk roads	183,737 ,,
Total . . .	310,421 ,,

Area of wearing course laid :

(i) Trunk roads	(*a*) 2 inches thick . .	1,061,959 square yards
	(*b*) 1½ inch ,, . .	nil ,, ,,
	(*c*) ¾ ,, ,, . .	22,467 ,, ,,
	(*d*) Miscellaneous . .	15,000 ,, ,,
	Total . . .	1,099,426 ,, ,,
(ii) Non-trunk roads	(*a*) 2 inches thick . .	1,317,941 square yards
	(*b*) 1½ inch ,, . .	1,860,777 ,, ,,
	(*c*) ¾ ,, ,, . .	35,216 ,, ,,
	(*d*) Miscellaneous . .	154,399 ,, ,,
	Total . . .	3,368,333 ,, ,,

Total : 4,467,759 square yards

Total quantity of base course and wearing course : 665,000 tons

RESULTS ACHIEVED

The results achieved in the four counties mentioned by the use of gravel asphalt during the 4-year period have been :—

(1) The restoration of a large mileage of roads—which were damaged by Government traffic during the war—has been completed in a much shorter time than would have been possible by any other means available.

(2) The work has been done for a capital expenditure approximately £200,000 less than if any other comparable form of construction

had been utilized. It is estimated that the next cheapest form of road-stone would have exceeded the cost of the gravel asphalt by about 6*s*. per ton (laid).

(3) A large saving in future maintenance expenditure will be realized, because the dense surfaces laid will not require surface-dressing as would the tar- or bitumen-macadam work which is popular in East Anglia, and which would no doubt have been utilized under normal conditions. The value of this saving is estimated at £263,491, assuming that no maintenance work is required for 10 years.

It seems not unlikely that the 10-year period may be exceeded, and in that event the saving would be correspondingly increased.

(4) The use of local material has resulted in a great saving of road and rail transport. The average distance of haul for the gravel asphalt between the pits and the road has been about 14 miles. The average distances from the next nearest sources of supply is about 104 miles. Thus the haulage of the 665,000 tons of asphalt laid has been about 90 miles less than if imported road-stone had been used. This is equal to a saving of some 59,851,620 ton-miles, neglecting the weight of the transport vehicles on both the laden and the return-empty journeys.

Viewed in a more practical manner, it represents a saving of, say, 1,663 special trains, each bringing 400 tons of road-stone 90 miles, or the equivalent (see *Fig. 2*).

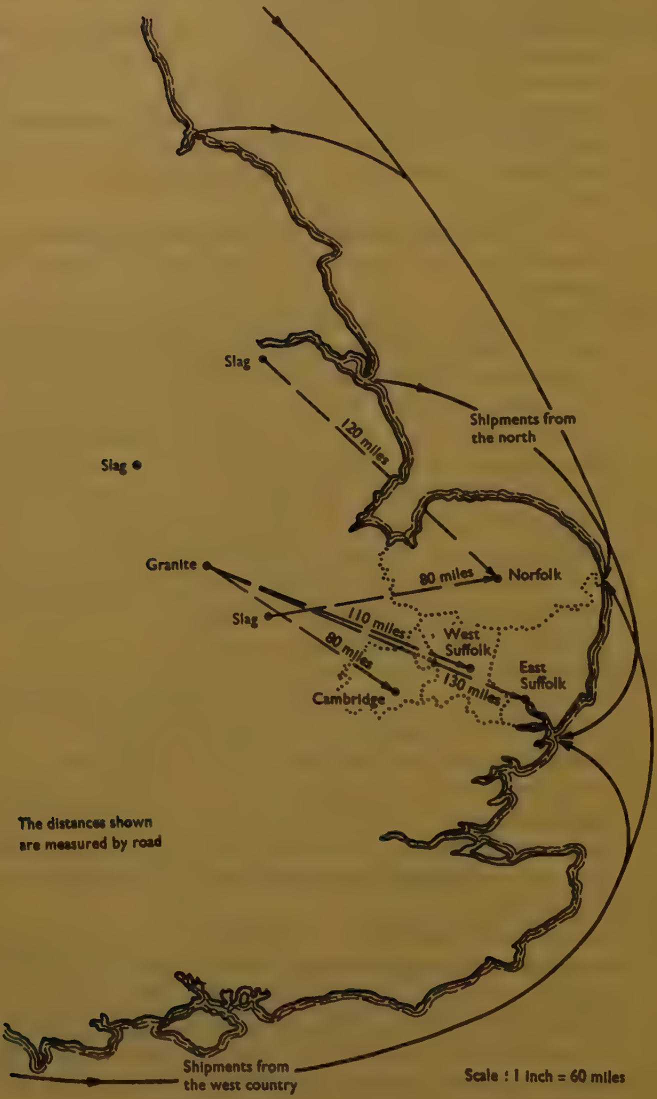
(5) The uninterrupted supply of mixed asphalt from the local plants has made it possible to make full use of Barber–Greene finishing machines for laying the surfaces. These machines, which were essential to the rapid completion of the programme of restoration work, could never have been used economically in the numbers required, if the road-stone had had to be hauled over long distances. The rate of supply would have been too slow and too intermittent under the conditions ruling when the work was done.

(6) A step forward has been made towards securing uniformity of surface—long recognized as a most desirable aim of highway engineers. In the four counties mentioned, on trunk roads at least, the driver no longer passes from one type of surface to another when he crosses the county boundary. He probably remains on rolled asphalt, so that his driving reaction to the road surface does not require adjustment.

Administrative Organization

When the gravel asphalt programme in the Eastern Division of the Ministry of Transport was started in 1946, it was first applied to trunk

Fig. 2



MAP SHOWING ROUTES FOR SUPPLY OF ROADSTONE TO EAST ANGLIA roads, which had suffered severe damage, particularly near the tank training areas.

At that time the Divisional Road Engineer had a large direct labour organization under his direction. It had been used during the war for executing road-works for the Service Departments, but this activity became much reduced in 1944 and 1945, so that the organization was available to help, as an emergency measure, with the repair of trunk roads in those areas where the damage had been greatest. The County Councils of Norfolk, East Suffolk, and West Suffolk found that all their own efforts at that time were needed to repair the hundreds of miles of badly damaged county roads, and they readily accepted temporary relief from maintaining the most troublesome lengths of trunk road.

The method which was then successfully adopted was to divide the work into two operations. First, the concrete foundation work, together with the kerbing and drainage, was performed by the Divisional Road Engineer's direct labour, and, secondly, asphalt contractors were employed to shape-up and re-surface the road, following along behind the direct labour gangs.

By that means a maximum rate of progress of about 1 mile of trunk road reconstruction per month was achieved at each site.

With the gradual return of more normal conditions, and with the present reduction in the sum of money annually available for road-works, the County Councils, as the Minister's Agents, have again taken over from the Divisional Road Engineer the responsibility for executing the whole of the trunk-road maintenance and improvement work. The Divisional Road Engineer's direct labour organization has therefore been disbanded. The procedure for surfacing by contract is, however, still being followed, the contracts now being let by the County Councils.

In letting contracts for the gravel-asphalt work, lengths of about 5 miles of 22-foot-carriageway road have been regarded as the optimum; this is equivalent to about 65,000 square yards. It is sufficiently large both to attract competition and to make it possible, if necessary, for a mixing plant to be brought to the site and erected without the cost unduly affecting the unit price. Similarly, when the work has been let, 65,000 square yards is enough to give the contractor a chance to organize his work properly, and it makes worth while the provision of good supervisory arrangements on the part of the Highway Authority. With small contracts there is sometimes a tendency to be lax in these matters.

For work on county roads much the same procedure has been followed, although the amount of preliminary work which it is necessary for the County Surveyor to do before the contractors can lay the surface has generally been very small. In rural areas it has usually consisted merely of cleaning the road, forming new turf kerb lines, up to which the asphalt can be laid, and making up the levels of the verges with filling. Any strengthening which the foundations were found to need has, with few exceptions, been done with base-course asphalt.

The asphalt surfacing on the county roads has been done by contract, generally of about the same magnitude as the trunk-road contracts.

Altogether nine different firms of contractors have been engaged, although most of the work has been carried out by five firms.

PRACTICAL CONSIDERATIONS GOVERNING THE WORK

The first condition which must be fulfilled if gravel asphalt is to be used economically is that a source of good clean gravel is available within a relatively short distance of the site of the road-work.

Apart from the increased cost which haulage from distant pits entails, it is undesirable for the asphalt to be too long *en route*, because undue loss of heat, especially in winter, may affect the laying and subsequent durability of the material.

The supply of gravel and sand must be sufficient to meet a continuous daily demand of 200 tons for asphalt purposes, if a single Barber–Greene finisher is to be kept working economically.

In East Anglia it has usually been possible, up to this time, to obtain gravel and sand within 20 miles or so of the road. The average haulage distance has not exceeded 14 miles, although a haul of about 30 miles has occasionally been made. Once, for a short time, a haul of 42 miles was undertaken. The location of the pits hitherto used is shown in *Fig. 1.*

A typical plant required for the complete local operation of winning the gravel and laying the asphalt on the road is shown diagrammatically in *Fig. 3*. It consists of :—

(1) The plant for winning, washing, and screening the gravel.

(2) Means of conveying the clean gravel from the screening plant to the drying and mixing plant—usually by lorry, dumper, or belt conveyor.

(3) Means of feeding the elevator to the drier—often a small excavator or shovel is used.

(4) The drying and mixing plant, which should have a capacity of not less than 20 tons per hour; 30 tons per hour, when practicable, is recommended. It is desirable that the mixer should work on the batch principle; in the Eastern Division the specification used excludes continuous mixers, because there is some doubt about the uniformity of the material they turn out.

In executing the programme described in the Paper the mixing plant has invariably been of the semi-portable type.

(5) Six-ton tipping lorries for hauling the mixed asphalt from the plant to the site where it is to be laid, the number needed depending upon the distance and the output. In the example illustrated in *Fig. 3*, six were used for a haul of 6½ miles.

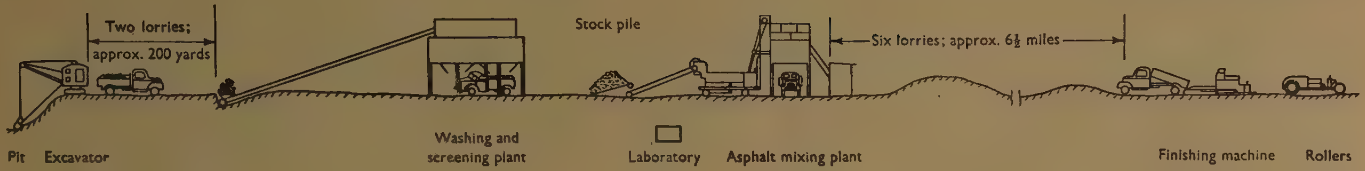
(6) The finishing machine for laying the asphalt. Hitherto, with a

Fig. 3

Number of men employed
- at asphalt mixing plant — 5
  - 1 foreman
  - 1 operator
  - 1 filler man
  - 1 lamp man
  - 1 man on boom scraper
- on lorries — 8 men — 8
- on road — 10
  - 1 foreman (assists raker)
  - 1 finisher operator
  - 2 roller drivers
  - 1 raker
  - 2 spreaders (pre-coated chippings)
  - 2 joint men
  - 1 labourer

Total 23

Rollers. – It is recommended that, for wearing-course work, two rollers, should be used: a 6-ton for rolling in the chippings behind the finishing machine, and an 8-ton for final consolidation and back-rolling



Manufacture and Laying of Asphalt with Local Aggregate at a Typical Plant single exception, Barber–Greene machines have been used. The exception was a Jaeger spreader box towed by a tractor, and was used for base-course work only.

(7) One or two 6-ton to 8-ton rollers. These should be capable of very rapid change from forward to reverse gear, and vice versa. Tandem rolls are probably the best, since the wide area of contact with the surface and the uniform distribution of pressure help in producing an even surface.

The output of the mixing-plant should, so far as possible, be correlated with the laying-capacity of the finishing machine, the number of lorries used to carry the material being regulated to suit. Hitherto the practice in East Anglia has been to use a mixing plant of about 200 tons per day nominal capacity. This is, however, short of the ideal, because the finishing machine, moving at its slowest speed of 7·85 feet per minute, can handle much more than this amount. The result is that too much stopping and re-starting of the machine has been necessary. Stopping tends to cause unevenness in the finished surface for three reasons :—

(1) The screed of the finisher may sink slightly into the hot surface while the machine is standing, and cause a depression which may not be entirely removed by subsequent rolling.

(2) The wave of material piled up before the roller, at the point where rolling has to stop temporarily, cools while the machine is standing, and cannot later be consolidated as thoroughly as the material on each side which is laid at the correct temperatures.

(3) Hot material remaining in the machine during a stoppage will cool, and perhaps partly set, so that it cannot afterwards be laid satisfactorily.

It is therefore most desirable that the finisher should be kept moving as continuously as possible and that the work should be organized so as to avoid unnecessary stoppages.

The order of working on a road of two traffic lanes is to lay the asphalt in two longitudinal strips, with a joint down the centre, one half being done in the morning and the other in the afternoon. Thus by nightfall the road should have been surfaced over the full width as far as a certain point along its length.

Traffic must be kept off the asphalt until it has cooled sufficiently to resist deformation ; this may require 1 or 2 hours in the winter and perhaps 2 or 3 hours in the heat of the summer. Afternoon traffic can usually be allowed to pass over the work laid in the morning.

The laying programme may be carried out by either of the following methods :

working alternate weeks on base-course and wearing course ; or laying the base course from end to end, and then the whole of the wearing course.

The advantage of the first method is that the base course is not required to carry traffic for long, so that damage to it ought not to occur; also, traffic does not suffer from having to traverse long lengths of temporary surface for periods of perhaps some weeks.

This method may also be advantageous when the gravel supply is deficient in $\frac{3}{4}$-inch aggregate for the wearing course, because an opportunity for accumulating $\frac{3}{4}$-inch material may occur during the periods when base course made with $1\frac{1}{2}$-inch aggregate is being laid. It has occasionally been necessary to change over from the manufacture of wearing course to base course for the reason that supplies of the $\frac{3}{4}$-inch stone have become exhausted and a period was required during which stocks could be replenished.

The advantage of the second method is that it enables the contractor to organize his work economically and without the breaks in continuity which must occur every time the mix is changed. With this method the base course may have to carry traffic for some time before it is covered with the wearing-course. During this period some displacement of stone may occur, which should be made good with a little fine material before the final surface is applied.

The total number of men required at a typical plant (shown in *Fig. 3*) for manufacturing the asphalt, transporting, and laying it at the rate of 200 tons per day is about twenty-three. This may be subject to slight variation to meet different conditions at other sites, but it can be taken as being typical of the average.

SUPERVISION

It was recognized from the commencement of the asphalt-surfacing programme in the Eastern Division that there must be adequate and competent supervision on the site. A Clerk of Works was therefore appointed as soon as the first trunk-road contract was let, and he was expected to devote the whole of his time to ensuring that the specification was complied with and to giving the contractors any co-operation and assistance which might be required in the production of good work.

As the amount of work in hand increased, and more contracts were let, an additional Clerk of Works had to be appointed for the trunk-road work.

The supervision of the work on county roads has varied somewhat in different counties. Some have followed the trunk-road practice of employing full-time Clerks of Works; others have made use of their existing staff, a procedure which may not always prove completely satisfactory, because there is sometimes a tendency to rely too much on inexperienced assistants.

The duties of the Clerks of Works have included: supervision of the mixing, haulage, and laying of the asphalt; the routine taking of samples of aggregate and mixed asphalt for analysis; temperature observations;

Fig. 4. Specimen Daily Inspection Report

TRUNK ROADS A.17 AND A.47 Ref: A.202/41/PR(Sub)

Contract closed down 5th to 11th March
(inclusive) due to snow and frost

MINISTRY OF TRANSPORT

ASPHALT CONTRACT DAILY INSPECTION REPORT

Contract No. *5*

Date *12th March, 1947* Weather *cold and dry*

Temperature *32° F. a.m.* *32° F. p.m.*

BINDER COURSE

Location of work *north side* from chainage *2,730* to *2,990* yards (approx.)
south side from chainage *2,730* to *2,990* „ „

Average depth *4½ inches* Average width *22 feet*

Tonnage laid *162 tons* (*cumulative total to date 3,656 tons*)

Samples taken:
none

Chainage	No. of sample	Position

WEARING COURSE*

Location of work *north side* from chainage............to............
south side from chainage............to............

Thickness of carpet measured at
north side

Chainage					
Thickness					

south side

Chainage					
Thickness					

Approximate area laid............square yards

Samples taken at chainage

Chainage	No. of sample	Position

REPORT ON GENERAL QUALITY OF MATERIAL AND WORKMANSHIP

(continued overleaf)

Satisfactory

Signed ....
Clerk of Works

Date.....................

* In this case, this portion of the Table is unused and is struck out by the Inspector.

the keeping of precise daily records in the form illustrated in *Fig. 4*; and the certifying of measurements of work done and of any extras ordered.

In the work executed under the direct control of the Divisional Road Engineer of the Ministry of Transport, the aim was to employ Clerks of Works who could take a full measure of responsibility, so that frequent visits to the sites by Engineers from Divisional Headquarters were not necessary. As an aid to this aim, the record sheets, which were sent daily to Headquarters by the Clerk of Works, have proved invaluable. They enabled an appreciation to be made, from a distance, of the progress of the work and revealed whether or not difficulties were being encountered. These reports are also of value in judging the subsequent performance of the surfaces, because they form a permanent record of the conditions under which the work was done and ensure that, should any local defects or peculiarities appear, detailed evidence is available which would enable the probable causes to be determined.

The Clerk of Works was assisted, where necessary, by a checker: a clerk employed primarily for the purpose of receiving the delivery notes for the asphalt. Whilst payment for the wearing-course material was made on the measured number of square yards, payment for base-course material had to be by the ton, because wide variations in thickness due to the irregular shape of the old carriageway made any attempt at superficial measurement valueless.

In general, there were no public weigh-bridges on the lorry routes between mixing plant and road, so a reliable means of recording tonnages without frequent checking had to be devised. This was possible because the material was all weighed at the mixing plant, the batches usually being exactly 1 ton each. Six batches were put into a lorry, making a 6-ton load which was recorded on the delivery note. Whilst this method may at first sight appear to give insufficient safeguard of correct delivery, the following precautions were taken and are believed to have been effective:—

The Clerk of Works, by frequent visits to the mixing plant, was able to satisfy himself that the batch-weights were correct.

The weighing machines on the batching plant were checked by the Weights and Measures Inspector of the County Council.

The checker inspected most lorry-loads to see that they were full.

Occasional lorries were directed, without previous warning, to the nearest public weigh-bridge.

Comparison of the contractors' delivery notes with the tickets obtained over a period from the public weigh-bridge showed that the aggregate total of the deliveries made indicated an average load of slightly more than the nominal 6 tons per load claimed for by the contractors.

Sampling and Testing

Samples of asphalt were taken regularly by the Clerk of Works in the presence of the contractor's Agent and sent to an independent analyst for examination; the results were recorded in the graphical form illustrated in *Fig. 5.* The contractors usually took duplicate samples at the same time, which they either analysed at their own laboratory, or retained for future analysis if the independent tests showed that the material failed to conform to specification.

The cost of analysis (bitumen extraction and grading of the aggregate) was £4 4*s.* per sample, so that regular daily analysis represented a monthly expenditure of about £100 per contract. In an effort to reduce this without loss of efficiency, the following procedure was devised and found to be reasonably satisfactory. On each of the first 3 days following commencement of manufacture of both base-course and wearing-course asphalt, samples were taken and despatched to the laboratory. By the fourth day the result of the first day's sample would be known, and if satisfactory a sample was not taken on the fourth day. If the second and third days' analyses were also satisfactory, sampling was thereafter done once a week, and continued so long as the results conformed to the specification. If the results were seriously outside the specification, analysis was again made daily until the mixing was shown to be correct.

Most contractors maintained their own laboratories, either on the site or elsewhere, and made regular analyses of the material they were manufacturing. In fact some of them maintained complete daily analysis records, and went to considerable trouble and expense to check that they were complying with the specification.

Aids to Measurement

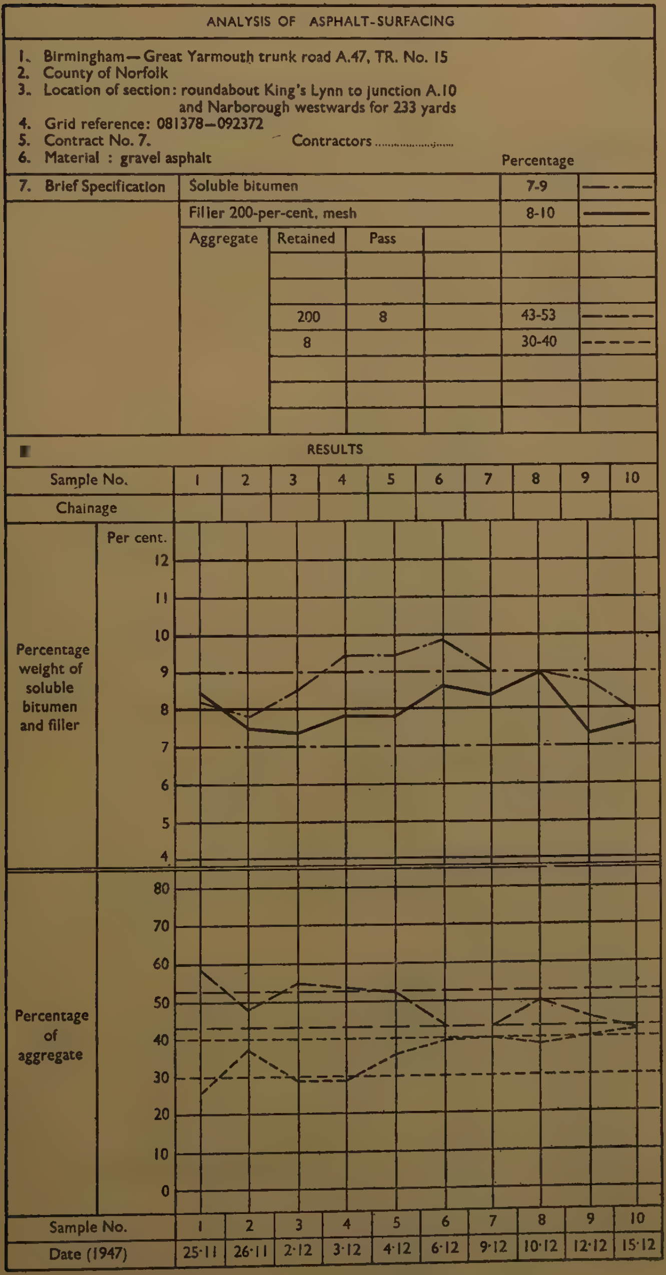
The Clerks of Works used two aids to measurement of which mention should be made, because they are both really essential if it is to be readily ensured that the requisite thickness of material is being laid and that the cross-fall and camber are correct.

The first was a small pocket gauge for inserting into the soft, newly-laid asphalt to measure the thickness (see *Figs 6*).

The second, known as a "cambergauge," is also illustrated in *Figs 6.* It is a light-weight bar, 12 feet long, one end of which rests on the kerb and the other end on the centre of the carriageway through the medium of an adjusting screw which shows the cross-fall as a direct reading when the bubble in the spirit-level is central. A travelling wheel with a calibrated arm to trail on the road surface can be passed along a rail on the bar and indicates the level of the road at every point on its width. The Whitley Council gave an Award to the Senior Clerk of Works who designed this instrument.

Three tests were applied to the finished surface. The first was a straight-edge test applied by the Clerk of Works immediately after the asphalt had been laid. It enabled him to bring to the notice of the contractor's Agent any variations of more than $\frac{3}{8}$ inch in level in a length of 10 feet, and to have them rectified.

Fig. 5



ANALYSIS OF ASPHALT-SURFACING

1. Birmingham—Great Yarmouth trunk road A.47, TR. No. 15
2. County of Norfolk
3. Location of section: roundabout King's Lynn to junction A.10
and Narborough westwards for 233 yards
4. Grid reference: 081378—092372
5. Contract No. 7. Contractors ....
6. Material : gravel asphalt

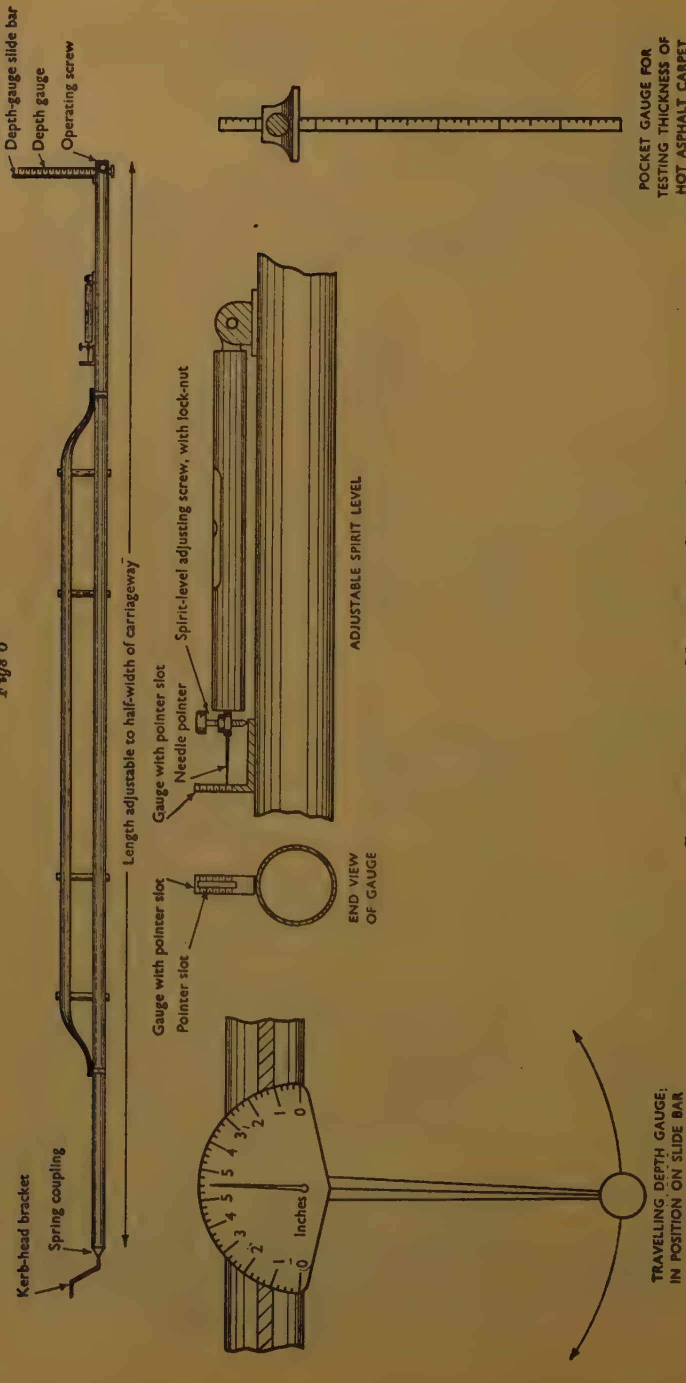
7. Brief Specification				Percentage	
	Soluble bitumen			7-9	—·—
	Filler 200-per-cent. mesh			8-10	——
	Aggregate	Retained	Pass		
		200	8	43-53	— — —
		8		30-40	- - - - -

RESULTS

Sample No.	1	2	3	4	5	6	7	8	9	10
Chainage										
Date (1947)	25·11	26·11	2·12	3·12	4·12	6·12	9·12	10·12	12·12	15·12

Record of Asphalt-Analysis Results

Figs 6



CAMBERGAUGE FOR MEASURING CROSS-FALL

The second is known as the "bumpometer" test, and consists in measuring the surface irregularities by means of the well-known single-wheel trailer drawn by a vehicle at a standard speed of 20 miles per hour. The upward movement of the wheel as it passes over bumps is recorded electrically, and is integrated. The distance travelled is also recorded, so that the machine gives a reading representing the inches of upward movement of the wheel in relation to the chassis per mile of road travelled.

Results of this test, taken at random from work done since the war, are shown in Table 1.

TABLE 1.—"BUMPOMETER" READINGS ON FOUR DIFFERENT TYPES OF SURFACE, TAKEN AT RANDOM

Surface	No. of surfaces tested	Bumpometer reading: inches per mile		
		Max.	Min.	Average
Trunk roads				
1. Gravel asphalt (machine laid)	10	175	130	152
2. Thin carpet coats, open or medium texture (some hand and some machine laid)	10	220	112	156
3. Concrete (uncovered)	9	205	130	167
4. Coated slag and coated granite surface-dressed (hand laid)	4	220	175	194
Non-trunk roads				
5. Gravel asphalt (machine laid) on class III roads	17	166	118	139

The class III road results are better than those for the trunk roads for two reasons :—

The whole width was surfaced with one pass of the machine, whereas the trunk roads had to be done in half-widths which made errors more probable.

The operators on the Barber–Greene machines tended to make more frequent adjustments of the levelling screws when working on trunk roads where the surfaces were laid to kerbs. On the class III roads, where there were no kerbs, the screws required less frequent adjustment and more even surfaces were therefore obtained.

Among the best results obtained in the Eastern Division has been a reading of 98 on gravel asphalt on a trunk road in Cambridgeshire; it is not included in the random results in Table 1.

The third series of tests applied have been skidding tests with the well-known motor-cycle combination run by the Road Research Laboratory. Those tests were originally started in order to provide an answer to the critics who said that gravel asphalt was a slippery surface for traffic. Some thirty lengths have been under observation since 1946 and the results for each are recorded in graphical form; a typical record is shown in *Fig. 7*. It will be noted that during the first 17 months after the surface had been

Fig. 7

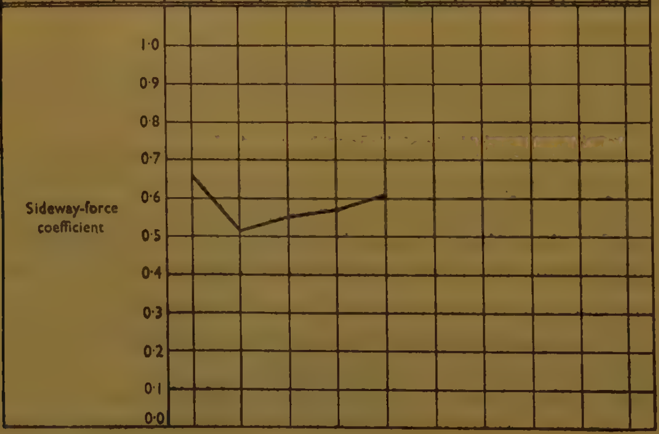
MEASUREMENT OF SIDEWAY-FORCE
COEFFICIENT ON CARRIAGEWAY SURFACE

London—Norwich trunk road A.11, TR. No. 2
County of West Suffolk
Location of section: immediately south-west of Icklingham—Eriswell crossroads
Reference point on map 3 : grid reference 206942
Type of surface: gravel asphalt
Brief specification: 2-inch gravel-asphalt surfacing, with ⅜-inch pre-coated chips at 80–100 square yards per ton

Laid by........................ Date: December 1945

RESULTS

Date		15.4.46	26.9.47	22.2.49	16.5.50	18.4.51					
Stopping distance: feet			52	51	47	44					
Sideway-force coefficient at m.p.h.	⑤	0·88	0·70	0·67	0·81	0·70					
	⑩	0·83	0·67	0·64	0·74	0·69					
	⑮	0·77	0·63	0·62	0·67	0·68					
	⑳	0·72	0·59	0·59	0·63	0·67					
	㉕	0·69	0·55	0·57	0·60						
	㉚	0·66	0·52	0·55	0·57	0·61					



RECORD OF MEASUREMENT OF SIDEWAY-FORCE COEFFICIENT laid the sideway-force coefficient fell from 0·66 to 0·51, but two subsequent tests made in 1949 and 1950 indicated that it had risen to about 0·57—well above the danger level of 0·3. That behaviour was typical of all the work done.

Specifications Used

The specification chosen, when the work was commenced in late 1945, was War Emergency British Standard No. 1152, 1944, entitled "Rolled Asphalt." However, a few small adjustments known to be desirable were made. The same mixtures, based on this specification, have been used for trunk and non-trunk roads.

Nearly all the work has been done with an asphaltic cement made with residual bitumen having a penetration of 60/80 at 25° C.

The composition of the base course has generally been :—

Soluble bitumen	5 to 6 per cent.
Aggregate passing $\frac{3}{16}$-inch-mesh sieve	25 to 35 ,, ,,
Coarse aggregate	55 to 75 ,, ,,

and of the wearing course :—

Soluble bitumen	7 to 9 per cent.
Aggregate passing 200-mesh B.S. sieve	8 to 10 ,, ,,
Aggregate retained on 200-mesh B.S. sieve but passing 8-mesh B.S. sieve	43 to 53 ,, ,,
Aggregate retained on 8-mesh B.S. sieve	30 to 40 ,, ,,

The wearing course has had $\frac{3}{4}$-inch pre-coated granite chippings rolled into the surface, which carries a 5-year free maintenance guarantee.

A number of variations have been made from time to time with the object of utilizing less expensive mixtures to suit special conditions. These variations are as follows :—

(1) Enriched base-course material used as running surface without further treatment

(*a*)	Asphaltic cement	6 to 7 per cent
	Filler (to pass 200-mesh B.S. sieve)	5 to 7 ,, ,,
	Sand	42·5 to 52·5 ,, ,,
	Gravel aggregate (graded $\frac{3}{4}$–$\frac{1}{8}$ inch)	35 to 45 ,, ,,
(*b*)	Asphaltic cement	5·5 to 6·5 ,, ,,
	Filler	4·5 to 6·5 ,, ,,
	Sand	33·5 to 43·5 ,, ,,
	Gravel aggregate ($\frac{3}{4}$–$\frac{1}{8}$ inch)	45 to 55 ,, ,,

Both (*a*) and (*b*) carry a 1-year free maintenance guarantee. Both have been used on trunk and non-trunk roads.

(2) Enriched base-course material surface-dressed

About 5 miles of this has been laid on a trunk road to the following composition:—

Soluble bitumen	5·5 to 6·5 per cent.
Aggregate passing 3/16-inch-mesh sieve .	35 to 50 „ „
Coarse aggregate (1½–3/16 inch)	45 to 60 „ „

It was surface dressed with ½-inch granite chippings spread over 90–100 square yards to the ton and with hot tar (E.V.T. 35°–38° C.) at 5½ square yards per gallon.

This surface, including the surface dressing, carries a 3-year free maintenance guarantee.

Variations (1) and (2) are both suitable for use where a considerable measure of re-shaping and side-strengthening is required, but where the amount of money which can be spent is relatively small. They are especially suitable when carrying out a stop-gap improvement until such time as complete reconstruction is practicable.

(3) Sand carpet used as wearing course

The specification for trunk-road work was:—

Asphaltic bitumen (180/220 penetration)	10 per cent.
Filler	5 „ „
Sand	85 „ „

The rate of spread was 37 square yards per ton and ⅝–½-inch pre-coated chippings were applied thickly. The period of guarantee was 3 years.

This material is relatively expensive, but is useful in towns and villages where existing kerb and step levels make it undesirable to raise the level of the carriageway by more than ½ or ¾ inch. It is insufficient to correct completely any bad irregularity in the surface.

(4) Medium-texture gravel carpet

Several miles of this have been laid on the lines indicated in B.S. 1241 but incorporating soluble bitumen (190/210 penetration) instead of the tar provided for in that specification. The composition was:—

Soluble bitumen	5 to 6 per cent.
Coarse aggregate	65 to 75 „ „
Fine aggregate	15 to 25 „ „
Filler	3 to 7 „ „

This carries a 2-year free maintenance guarantee. It has been used on trunk and non-trunk roads.

(5) Dense-tar carpets

Lengths of dense-tar carpet to the British Road Tar Association's specification, but with local gravel aggregate, have been laid on a trunk road and a class III road. The object was to obtain a comparison with gravel asphalt, both of manufacturing and laying technique, and of performance.

The composition of the base course was :—

1½-inch graded gravel	65·5 per cent. by weight
Sand (⅛ inch down)	29·0 ,, ,, ,, ,,
Tar binder	5·5 ,, ,, ,, ,,

The composition of the wearing course was :—

¾–⅛-inch graded gravel	34·7 per cent.
Sand (⅛ inch down)	45·8 ,, ,,
Filler	12·0 ,, ,,
Tar binder	7·5 ,, ,,

The tar used was a 49/50° C. E.V.T. type-“ B ” tar.

Apart from the pre-coated chippings, the aggregate used in all the mixtures mentioned above has been local gravel and sand.

Coarse aggregate.—The gravels used have been flint, rather mixed in shape and of smooth or pitted surface texture. The average water absorption has been about 2 per cent., the specific gravity about 2·5, and the aggregate crushing value about 16 (equivalent to 30,000 lb. per square inch). The grading has been reasonably uniform.

Fine aggregate.—The sand used has been clean pit sand, although tending to be rather coarser than was specified. On occasion, sands from two different pits have had to be blended in order to comply with the required grading, which is :—

	Percentage by weight	
	Min.	Max.
Passing 85-mesh and retained on 200-mesh sieve	20	40
Passing 36-mesh and retained on 85-mesh sieve	40	70
Passing 8-mesh and retained on 36-mesh sieve	6	12

There has been no difficulty in complying with the requirement that the sand should not contain more than 3 per cent. by weight of silt, loam, or clay.

Binder.—Residual bitumen has been used throughout, with the exception of a short length of trunk road in the City of Norwich, where the asphaltic cement was a 50-per-cent. mixture of residual bitumen and lake asphalt. This exception was made so that a comparison could be obtained of the non-skid properties of surfaces having the two different binders. It is accepted that lake asphalt is likely to produce a road offering better non-skid characteristics than will residual bitumen, and there will now be an opportunity of measuring this by periodic tests.

Filler.—Portland-cement filler has been preferred to ground limestone, because the experience has been that the latter may require rather greater care in mixing. Trouble has been experienced on three occasions, when using limestone in mixtures having a bitumen content near the higher end of the specified range, because the asphalt was slow to harden. The difficulty was examined at the Road Research Laboratory, where it was confirmed. It was found, however, that hardening is satisfactory so long as the bitumen content does not exceed about 7·75 per cent.

Similar trouble has not occurred with Portland-cement filler, which therefore appears to enable advantage to be taken of a wider specification for bitumen content.

COST OF GRAVEL ASPHALT COMPARED WITH OTHER ROAD-STONE

Perhaps the principal advantage which has been realized by the use of gravel asphalt in East Anglia is that it has been less costly to supply and lay than would have been any other available form of surface of equal durability.

The comparative costs shown in Table 2 are typical of the work done in this particular area. They are all actual contract prices and apply to the period 1947–1950. The prices all include those for laying, and the asphalt mixtures are to the specifications already described.

In comparing the cost of base-course asphalt with concrete for foundation work, the density of the asphalt is taken as being $\frac{18}{11}$ ton per cubic yard. The cost per cubic yard, based on the average price of 36*s*. 6*d*. per ton, is therefore 59*s*. 9*d*. This is rather less than the present price of concrete. In addition, cement, of which there has been a shortage, is not required for base-course material.

Cost of Reinstatement

It is desirable that all trenches opened during the maintenance period to enable service mains and cables to be laid should be reinstated by the contractors. The contract should therefore include a schedule of reinstatement charges, which are usually on a sliding scale. The following is a typical example from a recent contract:—

Minimum charge	50*s*.	
Up to 5 super yards	45*s*.	per super yard
Over 5 super yards and up to 10 super yards	35*s*.	,, ,, ,,
Over 10 and up to 15 super yards . .	25*s*.	,, ,, ,,
Over 15 and up to 20 super yards . .	20*s*.	,, ,, ,,
Over 20 and up to 50 super yards . .	17*s*. 6*d*.	,, ,, ,,
Over 50 super yards	15*s*.	,, ,, ,,

STUDY OF RESULTS OF WORK DONE

Wearing Properties

The oldest of the gravel-asphalt surfaces have now been in use for about 5 years, during which their maintenance has been the liability of the contractors. The amount of maintenance work required has so far, with the exception of one length of about ¼ mile, been negligible.

TABLE 2.—COMPARATIVE COSTS OF GRAVEL ASPHALT AND OTHER ROAD-STONE

Material	Price:			
	per ton		per sq. yd.	
	s.	*d.*	*s.*	*d.*
1. *Average prices for all work executed*				
(*a*) Gravel-asphalt base course	36	6		
(*b*) Gravel-asphalt wearing course (2 inches thick) * .	51	7½	4	11
2. *Prices of typical individual work*				
(*a*) Enriched base course used as running surface without pre-coated chippings; 2 inches thick, 10·6 square yards per ton	44	0	4	2
(*b*) E.O. on above for ¾-inch pre-coated granite chippings at 180 square yards per ton			0	4
(*c*) Enriched base-course mixture used as wearing course after surface dressing, average 3 inches thick (extra for surface dressing 6½*d*) . . .	38	9	—	
(*d*) Sand carpet used as wearing course (37 square yards per ton) with pre-coated chippings . .			from 2	10
			to 3	0
(*e*) Medium-texture bitumen-gravel carpet (16 square yards per ton)	42	0	2	7½
3. *1950 contract in Cambs. for gravel-asphalt and dense tar*				
(*a*) Gravel-asphalt base course	33	6½		
(*b*) Gravel-asphalt wearing course, 2 inches thick * .	46	2	4	4¾
(*c*) Dense tar and gravel base course	30	9		
(*d*) Dense tar and gravel wearing course, 2 inches thick *	45	1	4	3½
4. *Other surfaces*				
(*a*) Nominal 1¼-inch-thick coated macadam (Cambs. 1950), 19 square yards per ton	52	3	2	9
(*b*) 3-inch-thick coated macadam (annual contract):				
2¼-inch base course 6*s.* 6*d.* per square yard				
¾-inch wearing course 2*s.* 3*d.* „ „ „				
Total for 3-inch work	78	9	8	9
(*c*) 1-inch-thick coated macadam carpet coat (annual contract), 22 square yards per ton	61	0	2	9
(*d*) 3-inch-thick single-course coated macadam, including sealing, but not subsequent surface dressing (10 square yards per ton)	62	6	6	3

* Includes ¾-inch pre-coated granite chippings spread at about 110 square yards per ton.

One ton of wearing-course asphalt (including the weight of the pre-coated chippings) covers about 10·5 square yards to a thickness of about 2 inches.

The surfaces are all wearing well, although there has been some local loss of pre-coated granite chippings in places where they have been applied too liberally; the area so affected is not large.

With the wearing away of the bitumen by traffic, the gravel aggregate has become exposed, and has helped to make the roads relatively light in colour. The colour generally is especially pleasing where red-granite chippings have been used. It is suggested that care should be exercised in the choice of granite of not too dark a colour for the pre-coated chippings.

There has been very little loss of gravel from the surfaces, although a few of the exposed flints and pebbles have fractured and splintered owing to the passage of heavy traffic.

A rather more serious though local defect is the loss of pre-coated chippings. It is thought that the principal reason has been too heavy application, arising from a desire to produce a surface offering the maximum resistance to skidding. Hitherto, the specified rate of spread has been 80–100 square yards per ton. The revised British Standard for rolled asphalt required 70–130 square yards per ton of ¾-inch chippings. The Author's view is that even 130 square yards per ton is too close an application if loss of chippings is to be avoided as the surface wears. About 175 square yards per ton is probably a more satisfactory upper limit.

Cover of 100 square yards per ton would be very good if the chippings could in practice be spread uniformly, but unfortunately the fact that the coating of binder causes the stones to stick together results in bunches, which are difficult to break down, being rolled into the surface. The individual stones in the bunches then become loose because they have insufficient mortar round them. If the stones are to remain in place, it is obvious that they must be closely surrounded by mortar, and there is a better chance of this being achieved if the chippings are not spread too thickly.

It has been found, when laying dense-tar surfaces in Cambridgeshire, that the granite chippings do not stick together so much when coated with tar having an E.V.T. of about 50° C. as they do when coated with bitumen. It is therefore suggested that, if it is desired to spread the chippings really closely, the pre-coating should preferably be done with this tar binder.

A chipping which fractures under the roller may in time become displaced altogether. It is therefore important that the crushing effect of the roller on chippings of a given strength should be known. A few tests to discover this have been made by comparing the grading of the chippings before rolling with that after rolling. They indicate that approximately 7 per cent. of the chippings are broken during rolling, and that the effect is more pronounced when the chippings are closely spaced together. More investigation into this question is desirable.

The rate of spread of the chippings is also important because it affects the resistance of the surface to skidding. Extensive tests made in the Eastern Division with the help of the Road Research Laboratory show that a sideway-force coefficient of not less than 0·4 at 30 m.p.h. can be expected with a spread of 200 square yards per ton. The value for 100 square yards per ton is not less than 0·5.

A sideway-force coefficient of 0·4 is adequate on a road of good shape and alignment, and experience in the Eastern Division shows that accidents caused by skidding do not occur on such a surface.

An examination of the surfaces laid has been made in an attempt to determine what effect, if any, the crushing strength of the granite used for the pre-coated chippings has upon the numbers of chippings which become fractured and lost under traffic. The result is shown in *Fig. 8*, which also compares the chipping work done by different contractors. The relationship shown between crushing strength and fractured and displaced chippings is rather inconclusive, although it does appear from the diagram that the weaker granites do not give results quite equal to the tougher ones. It is suggested that there is scope for further investigation into this relationship. *Fig. 8* also shows that some contractors' chipping work has, on the whole, been much better than others.

Fig. 9 shows the result of an attempt to establish whether there is a direct relationship between the number of chippings which fracture under traffic and the number which become displaced. It appears that there probably is a relationship, and it is suggested that this point also is worth closer examination.

The final outcome of these examinations might conceivably be that the use of granite having a certain minimum crushing strength should be specified.

Another likely cause of loss of chippings from the surface is that the thickness of the coating of bitumen may be less than is specified, and less than is required to hold them in place.

A question which is often discussed, as affecting the wearing properties of the asphalt, is: what precautions should be taken, when laying the hot asphalt on old surfaces which are fat with binder from previous surface dressings, in order to ensure that the old binder will not find its way through to the top of the new asphalt? It will be remembered that the heat in the asphalt is usually sufficient to soften or melt the old binder.

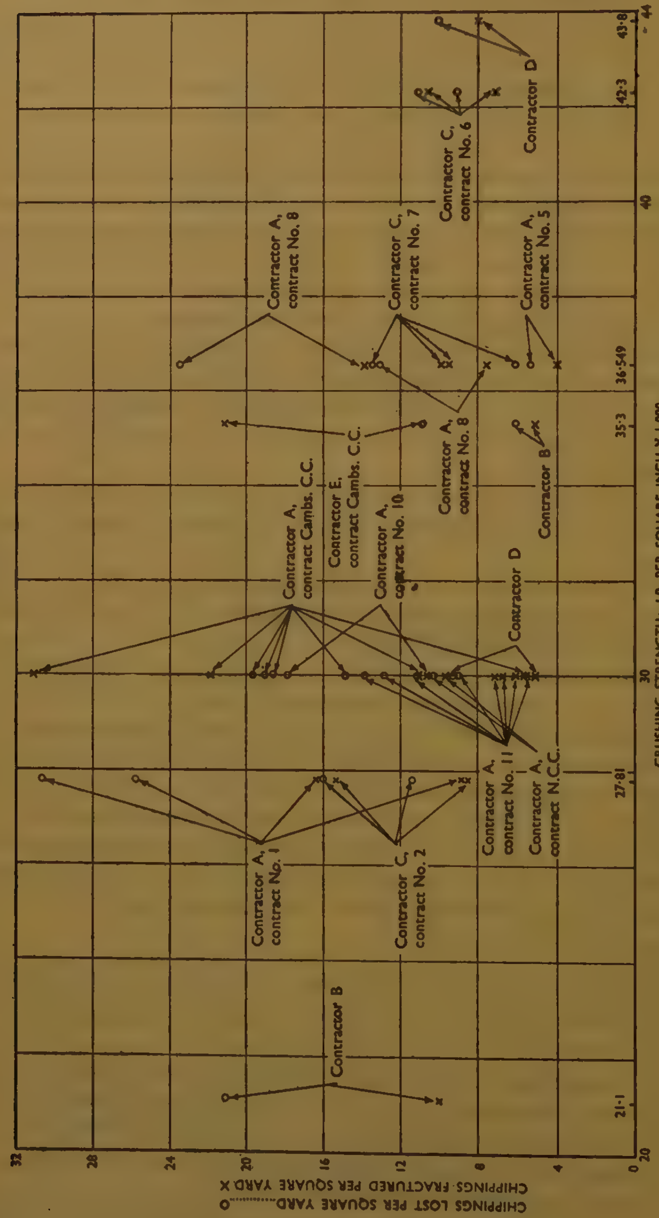
Much of the work done in the Eastern Division has been on roads which had an excess of old tar or bitumen on the surface, and the experience gained suggests that compliance with the following rules will yield satisfactory results:—

(1) However "fat" the old surface, there is no need for precautions if a base course, 1 inch or more in thickness, is provided under the wearing course.

(2) On a surface which is moderately fat or rich in binder, it is safe to lay a 2-inch wearing course direct, without precautions; but if the amount of old binder is very excessive it is desirable either to provide a layer of base-course asphalt, or, alternatively, to render the binder harmless by use of a burner.

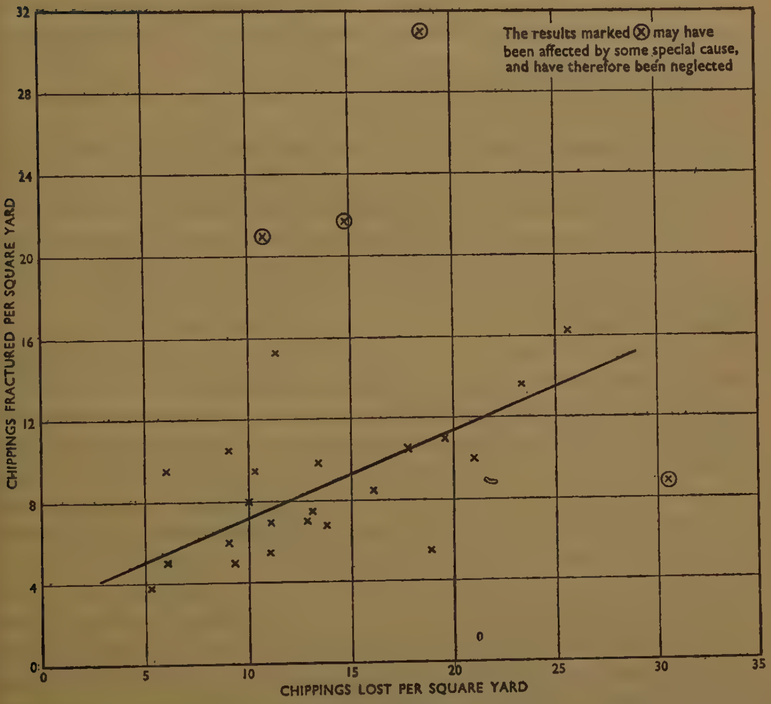
(3) A wearing course less than 2 inches thick may be laid direct on to old surfaces containing normal amounts of binder. There

Fig. 8



Behaviour of Pre-coated Granite Chippings Under Roller and Traffic

Fig. 9



The results marked ⊗ may have been affected by some special cause, and have therefore been neglected

Graph to Illustrate Relationship Between Chippings Lost and Chippings Fractured

would be risk, however, if the surface were at all fat, and precautions should then be taken.

In laying the asphalt directly on concrete or other dry surfaces it is desirable to apply a tack coat of bitumen emulsion, spread at about 16 square yards per gallon.

Gravel Asphalt on Concrete

In common with other forms of bitumen surfacing, a gravel-asphalt wearing course, when laid direct on to concrete roads of normal construction, may develop cracks immediately above the contraction and expansion joints in the concrete. This is to be expected because the adhesion between the two materials causes the asphalt to move with the greater mass of concrete as changes of temperature occur.

Several methods of dealing with this difficulty are being examined and tried. The first is the construction of the whole road in gravel asphalt—that is, by substituting a gravel-asphalt foundation for concrete. Since the cost of making and laying the asphalt is no more than that of concrete, this method seems attractive, provided that a greater thickness of base is not required than would be the case with concrete. This method has been, and is being, used in the Eastern Division, and further investigations are being made to enable information to be gained about the thickness of base-course asphalt required.

The second method which commends itself is that of constructing the concrete base, if it is desired to use concrete, of a very lean mix and low water/cement ratio, so that contraction cracks will be more or less eliminated. Expansion joints need only be very widely spaced and it is thought that asphalt superimposed upon this type of base would not develop cracks to any serious extent.

A third method is to provide reinforcement, top and bottom, in the concrete, in order to control cracking. Under present conditions, however, this will generally be found too costly to be practicable.

The problem is different where it is desired to lay asphalt on to an existing concrete surface which has not been specially constructed to receive the asphalt, and which has expansion and contraction joints at the normal spacings. No means have so far been found of preventing cracks in asphalt laid on this kind of concrete base, although the introduction of steel fabric reinforcement into the asphalt above the joint has been tried.

Where cracks have occurred in asphalt on concrete, the practice in making them good hitherto has been to cut a V-shaped chase, about 2 inches wide, in the asphalt and to refill with cold asphalt. This treatment appears to be moderately successful, especially if the remedy can be applied when the weather is not too warm. There is a tendency, however, for the crack to reappear in the cold asphalt filling, although it is then not so conspicuous as in the original surface. This method is rather expensive and has been abandoned in favour of sealing the crack with a bitumen-rubber mixture.

Reflecting Studs

When contracts are let for the laying of the gravel asphalt it is the practice to include an item for the provision and fixing of the reflector studs along the centre-line. It is considered desirable to allow the contractors to do this work, rather than that it should be done by the Council's Direct Labour, because the contractor has a 5-year maintenance guarantee and any cutting of the surface should be his responsibility so long as the guarantee is valid.

The contractor's price includes free re-fixing of any studs which become loose during the 5-year period, except for any which may break or fail as the result of fair wear and tear of the stud itself.

The usual method of fixing the studs is with hot sand-mix asphalt; cold asphalt is sometimes used, but is unsatisfactory.

SUGGESTIONS FOR FUTURE WORK

It is probably true to say that the development of hot-rolled gravel asphalt is the most notable episode in road-making practice which has been witnessed in East Anglia for many years. Some prejudice against it may still remain and it is perhaps desirable to examine the extent to which it is likely to be used in the future in that area as a principal constructional material.

So long as the present special need for economy in effort and expenditure persists, it will be essential that the least expensive material, drawn from the nearest sources of supply, should be used wherever possible. This is particularly so when it is borne in mind that the regular flow of supplies from the nearby plants enables the mechanical laying machines to be worked without the interruptions which might be expected if the road-stone were coming from a distant source. Local mixing is therefore a great practical advantage.

It may be that the suppliers of other raw aggregates will think it worth while to bring their dry material into the area and to mix locally. If so, they would be equally well placed for maintaining regular supplies. It seems doubtful, however, whether any other material can be made available at so low a price as the gravel, and it is expected that this will maintain its present advantage of cheapness, and therefore its popularity.

The dense material is likely to remain popular for the additional reason that maintenance costs are low compared with those of more open texture.

Medium-texture gravel carpets also, as described previously, are proving very useful, owing to their low first-cost; and they are being laid on all classes of roads where the more expensive type of surface cannot for the moment be provided for economic reasons.

The technique of mixing and laying the hot asphalt must, however, be improved so that the present defects can be eliminated. These defects are :—

(1) The pre-coated chipping work is not altogether satisfactory for the reasons already stated. It is suggested that the easiest remedy for this is more rigid control to ensure that the optimum number of chippings is obtained. The Author's opinion is that 150–175 square yards per ton of $\frac{3}{4}$-inch chippings is the present optimum, although a thicker application should be the aim as soon as a means of spreading the chippings evenly, and without bunching, is discovered. There is need for careful work on this problem until it is solved.

The best hope of eliminating the need for pre-coated chippings seems to be by increasing the stone content and developing for running surfaces mixtures of the kind already described under the title of "enriched base course." It is suggested that more such surfaces should be laid, on all classes of roads, and that some should contain mixtures of stone—for instance, gravel-granite and gravel-limestone—with a view to obtaining the best stability and non-skid characteristics without pre-coated chips. The cost of the work would be slightly higher than that of a 35-per-cent. gravel mix.

(2) The other defect which can be eliminated with care and good organization is that of unsatisfactory riding qualities. This, of course, is also associated with types of construction other than gravel asphalt, but the fact that the latter is laid hot increases the need for care. The remedy lies in adequate and competent supervision and in a thorough understanding by all concerned with the laying machine.

In particular the work should be organized, as it is in the county of Hampshire, in such a manner that stoppage of the machine, except at the mid-day break, is avoided, because it is the stoppages more than anything else which lead to surface irregularities. The organization necessary to achieve this may present a little difficulty, and it may slightly increase the cost, but it is well worth while.

The need for competent supervision cannot be stressed too much. In it lies the key to good results. Some highway authorities tend to be too lax in this respect and to be too ready to entrust the supervision to men having insufficient experience.

Work should not be started unless a fully-trained man is available to look after it as Clerk of Works, and he should be on or near the work all the time. The rate of progress of the laying work is so rapid that a defect which remains undetected for even 1 day can lead to a large area of new surface being unsatisfactory. The work should be examined as it is done and faults corrected as soon as they are discovered.

The laying temperature should be controlled, as well as the mixing temperature.

These last remarks may appear merely to be a statement of what should be obvious, but it is necessary to stress them because experience shows that first-class supervisory methods are by no means the rule.

It is suggested that, in order to meet the need of smaller authorities, who do not have sufficient work to justify the permanent engagement of a suitable man, a panel of experienced Clerks of Works might be established by one or other of the interested bodies. Highway authorities would then draw from the panel when they required assistance.

It is most desirable that a testing laboratory, properly equipped and staffed, should be available locally. If the samples have to be sent out to a commercial laboratory, the tendency is to analyse insufficient samples because the cost over the period of a contract can become substantial; the cost per sample at present is about £4 4*s*. If the highway authority has its own laboratory, it is to be expected that tests would be made daily on the spot, and the contractor might well be encouraged to participate in the routine testing.

It must be admitted that the arrangements already described for sampling and testing, whilst the best that could be devised, have not been completely satisfactory. There have been too many occasions on which the analysis has indicated a bitumen deficiency which the contractor has been able to refute on seemingly reliable evidence. There appears to be need for investigation into the probable reason for this difficulty, which has been of quite general occurrence.

Finally a word must be added about the dense tar surfaces with gravel aggregate which have been laid in Cambridgeshire—one on a trunk and the other on a class III road. The result on the class III road, as far as it can be judged after about 1 year, is good and equal to the asphalt. The trunk-road work, however, has given some trouble, which is probably connected with the weather conditions under which the work has been done.

There is a practical difficulty in the laying of this material, since the viscosity and temperature controls have to be so much finer than with bitumen. For this reason it is probably imprudent to use it where the following conditions cannot all be fulfilled :—

(1) The work should not be done in the depth of winter, nor under conditions of heavy rainfall.
(2) The mixing plant should be within ½ hour's travelling time of the site where the surface is to be laid.
(3) Qualified supervision of the mixing and laying process should be exercised continuously.

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The Paper is accompanied by four photographs and eight sheets of drawings, from some of which the Figures in the text have been prepared.

Discussion

Mr H. Bowdler observed that it was important, before considering the question of the laying of gravel asphalt on the minor roads in Norfolk, to know what the previous method of construction had been. In 1930, when the County had taken over those roads, they were in an appalling state and were little more than farm tracks on which local gravel had been placed from time to time and rolled in. An effort had been made to waterproof them by a very extensive programme of surface dressing, but no effort had been made to re-shape the roads until some years after when it had been considered necessary to scarify the surface, before applying a coat of slag rejects or flint to add strength to the road and surfacing with tarmacadam. The scarifying broke up the foundations and destroyed any bearing value which the old road might have had.

In 1946, when an enormous programme of restoration work had had to be faced, other available methods had had to be considered. The rail embargo had made it almost impossible to use slag; so far as flint was concerned, the embargo had been imposed by the contractors, because all their material went through the crushers, and hand-picked flints (the material used previously) were not available. Tarmacadam had frequently been delayed on the railway for 5 weeks or more and had had to be steamed or picked out of the wagons.

It had been clear that mechanization was essential. Had the prevailing rate of progress persisted, the work then scheduled would have taken 10 years—making no allowance for ordinary routine work. Before mechanization could proceed, however, there had to be plentiful supplies, and local gravel provided the necessary material for the manufacture of gravel asphalt in sufficient quantity.

This gravel asphalt had the big advantage that it fused with the old road surface and did not necessitate that breaking up which had previously been done, so that it had been possible to make use of the hard foundations in what existed of the old roads. By making a suitable grading it had been possible to get a mechanical lock and the material was considered to be more in the form of a concrete with a bituminous instead of a cement binder.

The early work had been on those roads which were badly shaped but which were fairly strong. It had been considered that the material was not very ductile and consequently should be laid only on stable foundations or hard well-consolidated material. Soon, however, it had become accepted practice to lay it on virgin soil to give added strength at the edge of the carriageways; it was also used for superelevation and for fills up to 12 inches or more. The misgivings had proved to be unfounded. The material had not, in fact, become brittle in the cold weather or too soft in the hot weather, and no deformation of the surfaces had been noticed.

They had cracked in one or two places where they had been laid on clay, but Mr Bowdler thought that that was to be expected.

The ductility was not confined to the hot weather, when it was possible to insert the blade of a penknife about 1½ inches into the material. That ductility was of great value in healing the cracks which occurred over concrete foundations, especially at the expansion joints. To illustrate that feature of ductility—which was an important one, because many people had criticized the material in the belief that it might become brittle—Mr Bowdler mentioned that in February 1951, after very heavy rain, a case had occurred where a mushroom-shaped hump developed in a gravel-asphalt surface. It was less than 30 inches in diameter and stood up from the surface of the road 8 inches. The traffic was kept off it, and when the water pressure had been released the surface went down; there was no cracking whatever and the surface was quite normal. That spoke well for the surfacing material.

Most of the material laid had been of a dense mix; that had been considered the most suitable for work on unclassified roads which were lightly trafficked. The Bituminous Gravel mentioned in the Paper was of open texture and likely to be damaged far more by frost and damp than by the effects of light traffic. The flint which Mr Bowdler had used was inclined to polish and fractured very easily, but the dense mix had overcome those difficulties almost entirely. The granite chippings which had been sprinkled on the surface of dense carpets to overcome any tendency to skid could be spread only fairly lightly on dense carpets, because of the high stone-content of the material, but on sand carpets ¾-inch granite pre-coated chippings had been spread at 90 square yards to the ton and excellent results had been obtained.

Mr Bowdler was surprised to see it stated in the Paper that the best riding qualities, obtained with the "bumpometer," had been on gravel asphalt, because he thought that the difficulties of laying gravel asphalt were far greater than those of laying cold material. The workability of the material varied so much with temperature that it was important to get continuity of supplies; otherwise there was a tendency for the last part of one batch to become cold, and the Barber-Greene operator had to tighten down his screed plate to try to iron that out, and when he got the next load at a higher temperature he had to adjust the screws again to release the pressure. That tended to make a wavy surface, a difficulty which was not experienced with the laying of cold material.

Comparative costs for asphalt had been referred to in the Paper, but Mr Bowdler did not think that they gave a true representation of the cost of the work. Shaping in gravel asphalt, the minimum size of stone used being ¾ inch, could be done for depths of 12 to 15 inches without any trouble at all. No deformation had taken place where what might be considered excessive depths had been used, and it could be feathered-out. If tarmacadam or any other type of shaping material were to be used, it was necessary to use a large material to get the proper interlock. The saving in cost by using gravel asphalt as compared with tarmacadam for a surfacing job was not so much in cost per ton of the material as in the saving of the material; feathering-out could result in large savings.

Gravel asphalt used as foundation material had engendered so much confidence that a number of roads had been constructed with that material as foundation, and several of them were carrying very heavy traffic. Most of the haunching work in the county was done in that way. It almost eliminated watching and lighting and its interference with traffic was almost negligible; the traffic could run on the material within an hour or so of laying. Hand pitching was, he thought, almost doomed, and it might be that gravel asphalt was going to take the place of the old hand work.

Mr H. C. Husband thought that the Paper would prove to be a valuable work of reference on gravel asphalt; he did not know of anywhere else where most of the information given in the Paper could be obtained.

His own immediate interest was the desire to explore the possibilities of using cheaper local materials in place of more costly imported ones for road work and large paved areas, particularly in connexion with some military establishments with which his firm was concerned at the present time. At those sites they had gravel available; the subsoil consisted of gravel, hoggin, or ballast, and the greatest possible economy had to be exercised. He did not think that anyone could seriously dispute the importance of using a local material rather than importing stone from 50 to 100 miles away, as would be the case in East Anglia.

Mr Bowdler had emphasized that gravel asphalt was very useful for base courses; that it had been used for forming superelevation in considerable thicknesses; and that it should be seriously considered for replacing pitching—with all of which Mr Husband would agree. In the Paper it was stated that the use of concrete had been considered; there was, however, a cement shortage and that had weighed heavily in favour of using a bituminous binder. Could the Author give similar information, for the purposes of comparison, about the use of Portland cement and bituminous materials as binders, and also mixtures of the two? There was a small quantity of cement in the Author's gravel-asphalt base-course, but Mr Husband had in mind mixtures in which the cement would have the higher proportion.

He presumed that less preparation of the aggregates would be required when using a cement binder. He had had some experience during the war of gravel asphalts for airfield surfacing in much the same part of the country as that with which the Paper dealt, and, as the Author had pointed out, it had been necessary to prepare and dry the aggregates carefully and watch the grading. Mr Husband appreciated the advantage, mentioned by Mr Bowdler, of the greater ductility which was likely to be obtained with a bituminous binder. From his own extensive motoring experience, he thought that the roads which had been built in Great Britain—including the Norfolk roads referred to in the Paper—were as well surfaced as any roads in any other part of the world, but it was very noticeable when one came to a section which had been laid with the Barber-Greene plant, and he wondered whether growing use of the Barber-Greene plant for asphalt roads was perhaps tending to retard the development of mechanical methods of concrete placing and soil stabilization using cement mixtures.

He had recently had an opportunity of inspecting roads laid at West Drayton. The comparison was not quite fair, because they were new roads, and it was not a matter of reconstructing old roads or laying the material on existing bases; but the roads which had been built at West Drayton under the direction of Mr W. T. Morgan, the Surveyor to the Yiewsley and West Drayton Urban District Council, in conjunction with the Road Research Laboratory, impressed him very greatly by their quality and cheapness, and he had seen them carrying heavy traffic during the building of large housing estates. There the local ballast or hoggin had been stabilized with Portland cement, largely mixed in situ, and surfaced with a thin carpet of gravel asphalt, making an excellent road very cheaply indeed, and following what he supposed was quite common practice in America.

He understood that the Author was now concerned with building a new Class 1 road in East Anglia, where the construction was entirely of cement-stabilized hoggin and the surface was gravel asphalt. Mr Husband suggested that information on that work would form a very useful extension to the present Paper and would give a valuable comparison. Whenever a new job was tackled it was very important to have comparative information, rather than information dealing with only one material. The Author was in the very fortunate position of not being biased; he was employing two methods of using local materials for road construction, and there seemed to be a wonderful opportunity to make a comparison.

Mr T. W. Tebbutt observed that the Paper might give the impression that gravel asphalt had been used on a large scale only in East Anglia. That was perhaps largely true, but it should be pointed out that it had been employed in other counties, notably in Hertfordshire, where from 1930 to 1950 more than half a million tons of gravel asphalt had in fact been laid—a considerable tonnage. That material had also been used with great success on airfields, particularly during the war, when it had been so important to save supplies of petrol and rubber tires. His own firm alone had laid 250,000 tons or more of the material as surfacing on airfields in East Anglia and elsewhere, and that represented an area of more than 5,000,000 square yards. A number of other contractors also had laid large areas of the same material, so that the total tonnage and yardage must have been very great indeed, and it performed a very good service to the community.

Mr Tebbutt agreed in most cases with the Author's conclusions regarding the non-skid properties of gravel asphalt. The graph in *Fig. 7* clearly showed that after 18 months' wear the surfacing became less slippery, and the Author in his opening remarks had added further information which bore that out. Mr Tebbutt thought that that tendency was particularly applicable to gravel asphalt, and from observations which he had made over a long period of years he had noticed that with gravel asphalt a pitted surface was obtained which continued to be less slippery than that provided by other forms of asphalt, and it improved as it wore. That was a tendency which perhaps had not been noticed as much as it should have been, and he thought that it was largely due to the fact that the gravel stone, being of a smooth nature, did not adhere to the mortar of sand, filler, and bitumen in which it rested so well as did an igneous rock. The result of that was that, while the mortar had a sufficient affinity to the gravel stone to hold the large particles, the small particles tended to become dislodged from the surface under traffic and a pitted or pockmarked surface was obtained. He thought that that was the reason why it retained its non-skid properties, or rather gained in non-skid properties, as it wore, whereas other forms of asphalt did not do so.

A very good example of this could be found in some gravel asphalt laid by his firm in Birdcage Walk, not very far from the Institution building, in 1934. It was a 2-inch carpet laid on a waterbound foundation without any base course, and he thought that those who examined it would agree that it was doing a good job, and they would see very good evidence of the pockmarked surface to which he had referred. There were a few cracks in it, caused by failures in the foundation, and it should be remembered that the work had been done a long time ago.

Reference was made in the Paper to the desirability, and almost the necessity, of having 200 tons a day in order to make a successful job. He agreed that it was an advantage to get 200 tons, or more if possible, but he would point out that there had been a great many jobs done with an output of less than 200 tons a day and in which the job had been quite good. There was room for improvement in anything, but he did not think it should be affirmed that it was not possible to do a good job if the output was less than 200 tons a day. It should also be borne in mind that 150 tons at 1½ inches thick would cover as much ground as 200 tons at 2 inches thick, and, since the area covered was the factor under discussion, it did depend on the thickness to which the carpet was being laid.

In trying to get as high an output as possible, it was often found that the application of the pre-coated chippings was the limiting factor in the amount of work which could be done in a day, and that led him to ask "why use chippings at all?" He thought, from his observation of the effect of wear on gravel asphalt, that, as with other forms of asphalt, it was possible to produce a form of gravel asphalt, with a high stone content which did not require chips, and he thought that that could be designed and used in such a way that it would be sufficiently non-skid to be safe when first laid, and it would improve as wear took place. He considered that every encouragement should be given to that aspect of surfacing work, in order that mechanization should be as complete as possible. On an average job it would be necessary to use double the number of men (if chips were put on) that would be required if no chips were put on at all.

Several references had been made in the discussion to the economies which could be gained by the use of gravel asphalt, because it utilized local supplies of material. Mr Tebbutt felt that it was necessary to sound a warning note there, because, owing to the Town and Country Planning regulations, it was now very difficult to get permission to open new gravel pits, so that there was already, in many of the areas which had been mentioned, an acute shortage of gravel. In the old days that would have been overcome by people getting permission quickly to operate new deposits. He felt that unless there was some speeding up of the process of dealing with applications, and also with objections, there would be shortages which would result in the material having to be brought long distances, with consequent expenditure of petrol which could be ill afforded. That was a very serious matter, and one which should receive the attention of all competent authorities.

The Chairman remarked that he had no fault to find with the specification in the Paper, which was very similar to B.S.594, but it was, of course, low in stone content. All specifications involving the use of gravel with which he had been concerned had to be largely a mortar with the stone content in the form of " plums " in the mortar. That was the correct way to use that kind of material, and for that reason he entirely agreed with the specification; the aim, however, of present-day road makers was to lay a surface which, from the very beginning, was non-skid, and that, in most cases, was possible only with a large stone content. The material under discussion had not a large stone content and the mortar was such that it was likely to become slippery; the result was that it had to be treated with chippings to make it non-skid. He did not know how many of the chippings were left on the roads now but he imagined the fact they were still non-skid was due mainly to the presence of those chippings which were left. That, however, was not the sort of non-skid surface that road makers were aiming at. In his opinion, a mixture with so low a stone content as the one under discussion and a mortar of the composition indicated, would not yield a non-skid surface and it would always be necessary to treat it.

Dr A. R. Lee said that the Paper was a very full account of the work done in East Anglia in developing gravel asphalt as a road-surfacing material. In comparison with American practice, he said, it was a little rare for engineers in Great Britain to write Papers on surfacing work for which they had been responsible. Such records of costs and progress, to which at some later time was added a statement of performance and life, provided the basis of an accurate assessment of the relative economic merits of different kinds of constructional materials, and he thought that there ought to be more papers of that type.

As Chairman of the Drafting Committee of the British Standards Institution which had formulated the present British Standard for gravel asphalt, Dr Lee was glad to see from the Author's remarks that after 5 years' wear the first material laid was so far giving excellent results. There was no reason why the material should not wear as long as other rolled asphalts in which the coarse aggregate was granite or limestone, for, after all, the basis of the material was a sand asphalt, and so long as that constituted the bulk of the material the replacement of the usual 30 to 40 per cent of the coarse granite by gravel should not introduce any major change in the properties of the asphalt. In fact, he saw no reason why gravel asphalt should not have a useful life of 20 years on the roads in East Anglia, provided its composition were uniform.

The dimensions of the work described were impressive—83 miles of trunk road and more than 450 miles of secondary roads—and it brought to mind the opportunities which were afforded by work of such dimensions. It was common, of course, in large civil engineering works for some part of investigatory work to be included in the job. It was essential in some cases to include such work to solve the immediate problems that were associated with the job; in other cases investigatory work might be included to deal with more general problems. Had consideration been given, in the present case, to the possibility of planning the work so as to include investigations required for the solution of some of the many problems in this field of highway engineering? At the present time, when financial stringency was so rigorously applied to all aspects of road construction and maintenance it was particularly important for the most to be made of the few opportunities that arose, and the period of waiting that was inevitable before results were obtained from experimental road work emphasized the need to take advantage of the opportunities presented by such work as described by the Author. There were a number of problems that came immediately to mind in the present case. It would have been useful, for instance, to have a direct comparison on the same piece of road, of the gravel asphalt and normal asphalt with granite, so as to have a direct comparison of their performance. Another question was the effect of the nature of the gravel. When the details of the present British Standard were being discussed, this question occupied many hours of discussion. In the work described, some variations of that type could have been introduced. Again, there was the question of the effect of increasing the stone content. The Chairman had suggested that by increasing the stone content a material might eventually be obtained in which there would be no need for pre-coated chippings.

What was the effect of changing the bitumen content? The composition now used was that given for the middle schedule of the present specification, but in East Anglia, where the climate was relatively dry and equable, it seemed to Dr Lee that there would have been a good opportunity to see whether a lower bitumen content would have given equally good results. That was a problem which required investigation; how wide was it possible to make the variation in the bitumen content for asphalt on roads such as those described, where the maximum traffic intensity was of the order of 2,500 vehicles a day or less, which would generally be regarded as medium to light traffic? Also, what would be the effect of a softer bitumen?

Another important problem was how gravel asphalt compared with gravel macadam, the gravel carpet. Reference had been made to some material of that type being laid. Had it been laid under comparable conditions? That was a point which should receive quite detailed consideration, because there was certainly a much lower cost given for the gravel carpet, and it looked as though it might be a serious competitor. He thought that a gravel carpet on those roads, if it were laid to the composition recommended in Road Research Bulletin No. 5, which had been issued some years ago, might have a life of as much as 10 years before any surface treatment would be required. He was not quite clear what were the factors that had influenced the decision to use the gravel asphalt in preference to the gravel carpet, and he would be glad if the Author would say a little more about that.

Finally there was the question of control. The Author had emphasized, in his opening remarks, the importance of control and the need for improvement there. Dr Lee had been astonished to see the economies which had been made on that aspect of the work. It would appear that in each 5-day week one sample of 2 lb. was the only check made on 2,400,000 lb. of asphalt. The Author was concerned about the inadequacy of those arrangements, and he would be clearly justified when it was appreciated that the specification required that the composition of the asphalt should be that found by analysis. It might be suggested that 0·5 per cent on the cost of material would not be an unreasonable charge to make for the cost of control. Such a charge would not be exceeded on the output in question by the cost of employing a full-time analyst working in a laboratory on the site. Rapid and reliable methods of analysis had been specially developed for that purpose and were in regular use by some manufacturers, and he would be glad to help the Author in that matter.

The question of control was also tied up with that of the type of asphalt plant used. The idea of using the continuous plant had been dismissed in the present case, and he understood that it had been dismissed because troubles had occurred at some time in the past; but a job of the kind in question, where one material was being manufactured on a very large scale, provided the optimum conditions for a continuous plant, if there were adequate control running with it. There seemed to be an opportunity for investigating how far a continuous plant would give a satisfactory and probably cheaper material if a control laboratory were on the site.

In conclusion he would be glad to know whether consideration had been given, in planning the work, to the possibility of introducing investigations for providing answers to some of these problems.

Dr R. S. Millard observed that the Paper described a fine example of public enterprise, which might well be followed in other parts of Great Britain, and applied wherever there were local materials which were not being fully exploited. The evidence which was being accumulated at the Road Research Laboratory from experiments in different parts of the country indicated that almost all types of aggregate could be used to make bituminous surfacing, provided the specification was chosen to suit the particular aggregate, the local climate, and the intensity of traffic on the road. Such aggregates as gravel and limestone, which had been looked at askance by some people in the past, could be successfully used with the proper specification, and all that was now necessary was the courage and enterprise to apply that knowledge.

The efficient use of the machinery did require that each job should be of reasonable size. The Author had mentioned 5 miles of road as the optimum size of contract. Dr Millard hoped that the Author had meant that it was the minimum for the efficient use of plant. Perhaps that was what the Author did mean, the upper limit being set at present by lack of sufficient money to make the contracts larger.

The Author had mentioned 200 tons per day as the amount of material required to keep the Barber-Greene going, and had suggested that it would be useful to have Barber-Greene machines with a lower bottom gear. Dr Millard thought, on the contrary, that it ought to be possible to use the Barber-Greene machines as they were intended to be used, in order to save man-power, and that the object should be to use materials which would enable advantage to be taken of the high laying speeds which the Barber-Greene could achieve. It should be possible to use two, or even three, asphalt plants and keep the spreading machine going, laying 400 or perhaps 500 tons per day. That might not be possible with the asphalts which had been discussed that evening, or with any dense high-temperature mixtures, but it should be quite possible with the coated macadams to which the Author had referred; and the increased output possible with them suggested that perhaps more use might be made of them, thus further reducing the costs below those given by the Author, and at the same time, by ensuring a constant supply of material, possibly improving the riding quality of the roads.

Dr Millard had been interested in the diagrams of the cambergauge and of the travelling depth gauge (*Figs 6*). In measuring longitudinal profiles the straight-edge had been used, and the maximum permitted variation in the 10-foot straight-edge was $\frac{3}{8}$ inch. That, of course, was the maximum permitted tolerance in the British Standard for rolled asphalt.

At the Road Research Laboratory some measurements had been taken to compare straight-edge records with the results of bumpometer and profilometer readings, and it had been found that if a maximum depression of $\frac{1}{4}$ inch or $\frac{5}{16}$ inch were accepted on a 10-foot profile—rather less than the maximum tolerance permitted in the Standard—the index of irregularity obtained on a bump integrator would be between 160 and 190 inches per mile, giving only a fair riding surface. To obtain a good riding surface, with an index of 130 inches per mile or less on the bump integrator, it would appear that the maximum tolerance which could be allowed on a 10-foot straight-edge was between $\frac{1}{8}$ and $\frac{3}{16}$ inch. Those figures were being considered by the Institution's Committee for use to give guidance in the forthcoming Code of Practice.

When he had first read the Paper he had been particularly struck by the enriched base-course material, to which reference was made, but it was discouraging that the contractors had only enough confidence in that material to guarantee it for 1 year. The material seemed to have distinct possibilities, being cheap and relatively easy to lay, and perhaps making it possible to dispense with the perpetual nuisance of pre-coated chippings. The Author, in his suggestions for future work, had suggested that more of that single-course material should be laid, and the results of those trials would be most interesting. There was still the question of the 1-year guarantee, however, and Dr Millard asked if the Author could say, from work already done, how well the enriched base-course material was, in fact, wearing.

Dr Lee had already mentioned the question of control, and had drawn attention to the need for, and the value of, increased control on work of this type. Generally both the authority and the contractor carried out independent analyses. That seemed to be an unnecessary duplication of effort. Could not a small testing laboratory be required in the contract on all jobs of £15,000 or more; or could not the Ministry or the Local Authority provide laboratories, possibly hiring them out to smaller local authorities that could not afford one themselves. If that could be done, the advantages were manifest. Instead of having to wait 4 days for the analysis, the results could be obtained, making use of such rapid methods as the Road Research Laboratory had developed, within an hour or so of mixing; and the contractor mixing; and the Contractor and the Clerk of Works could then both accept the result on the spot, or, if either of them expressed any doubt, it would be easy to do a rapid check straight away, and then they could both work together to remove any of the faults which might be revealed. That was the real use of control, to enable faults to be shown up very quickly and to ensure that a constant supply of uniform material was maintained. That put rather more responsibility on the Clerk of Works, but it was one which the Clerk of Works would be quite ready to carry.

The Author had suggested a panel of experienced Clerks of Works, and Dr Millard thought that was an excellent scheme. Not only would it provide a pool from which experienced men could be drawn but it could be a means of improving the status and ability of Clerks of Works.

Fig. 4 was a facsimile of the Daily Report submitted by the Clerk of Works. Those records were not only of day-to-day value in showing people at head office how the job was going, but of value as a permanent record. If details of analysis results could be included on those forms it would be possible, 10 or 15 years later, to go down the road with those records in hand and find out how each variation in composition had affected the performance of the material; and then—if at that time there was any gravel left to make gravel asphalt—it would be possible to draw up specifications based on the best materials which were now being laid.

Mr Robert Lacey said that nobody would disagree with the Author's emphasis on the importance of adequate and competent supervision, but he should, in Mr Lacey's view, at the same time have laid greater stress on the need for the employment, in all phases of the mixing and laying process, of skilled and experienced workmen. Gravel asphalt, like other asphalts, had to be laid correctly in the first place because if that were not done it was impossible to produce a good running surface by subsequent remedial measures. The statement at the foot of p. 174 of the Paper might therefore be misleading. The Clerk of Works could not detect surface irregularities by means of a straight-edge until the final rolling had been completed, and it was then too late to rectify the faults except by removing the asphalt completely and starting afresh.

The production of an even running surface on the wearing course depended very largely on the care exercised in the laying of the base course. If the base course were laid in patches, for the purpose of correcting the shape of the existing road, it was important that all the joints between the base-course material and the old road surface should be tapered with asphalt of a small gauge and that any damage caused to the base course by traffic should be made good before the wearing course was laid; otherwise the irregularities in the base course were reproduced in the wearing surface. The Barber-Greene finisher could not be relied upon to eliminate those irregularities in the process of laying the wearing course.

The Author had rightly drawn attention to the importance of keeping the Barber-Greene finisher moving continuously. That was often difficult to achieve in practice and it would be of considerable assistance in that direction if the machine were fitted with a lower gear to enable it to travel at speeds slower than its present minimum. Consistency in the composition of the material and in the laying temperature were also both vital factors in the production of a good surface.

The key to good results, however, lay primarily in the skill of the workmen. There were, among the contractors' personnel engaged on the programme of works in East Anglia, two or three foremen and gangs of workmen who had shown themselves to possess outstanding ability.

Where the standard of workmanship had been high it had invariably been due to the skill of those men rather than to the keenness of the supervision on the part of the employing authority. The Author had suggested the establishment of a panel of trained Clerks of Works; but a greater contribution to the improvement of the quality of future work would be made if arrangements could be made, within the industry, which would enable those competent and experienced foremen and operators to train others.

In Cambridgeshire, gravel asphalt was to an increasing extent taking the place of concrete for the construction of haunches, and of new carriageway foundations, as well as for reshaping existing roads. Base-course material 9 inches thick had been used for the foundations of class I roads in the county, and although it was too early to express a definite opinion on its behaviour it appeared so far to be entirely satisfactory. That method of construction was quicker than the use of concrete, was slightly less costly, and it required fewer men which, in view of the acute shortage of labour, was a distinct advantage.

It was hoped that gravel asphalt would last for many years, but how did the Author consider the surface should eventually be renovated? Would it involve the removal of the whole of the wearing course and the laying of a new course 2 inches thick, or was it possible to plane off, say, ½ inch, and to lay a fresh wearing surface?

Mr B. F. J. Bradbeer considered that appreciation of the Paper by road engineers would probably be measured according to the availability of road surfacing materials in their own areas. The use of unconventional local materials for base course or surfacing was very attractive at the present time, provided that such materials were within reach, and that the planning authority did not discourage the winning of them, which was one of the obstacles often encountered. Unfortunately, what was almost waste material in one area might be no cheaper than a conventional commercial product if haulage to another area, and possible stockpiling and handling, had to be taken into account. The rapid increase in cost due to haulage was a very great disadvantage in an area such as East Sussex, where local materials were scarce.

That was especially a matter of concern at the present time, because of the increasing use of mechanical surfacing plant. One of the problems which the Barber-Greene machine brought with it was an increase in the average speed of vehicles, and that immediately outmoded all the bends and superelevations, and much of the original shape of the road. There was a great field for the use of the low-grade aggregates under discussion in the preliminary re-shaping which was essential to provide a correct running surface. There were instances which he could give of where that class of material, using the cheapest possible aggregate, had been laid in re-shaping work to a thickness of 2 feet 3 inches on a trunk road which had been found to carry as many as seventeen thousand vehicles a day, and that had been down for from 3 to 4 years in more than one place without any movement whatever.

Would the Author comment upon the extent of the re-shaping which had been involved in the programme in East Anglia, and would he say to what extent the figures in Table 1 of the Paper referred to that programme? It was not clear whether all, or only some part, of the tabulated information referred to the programme under review.

Mr Bradbeer sympathized with what Dr Millard had said, namely that in the present British Standard the tolerance of $\frac{3}{8}$ inch in a 10-foot straight-edge was too low for the class of work which was now being done and which was expected from machine-laid work. Other engineers would share that opinion.

The human angle was very important. All manner of tests and inspections might be applied, but, from Mr Bradbeer's experience of handling labour, severity would not get the best results. Good workmanship was the result of leadership and interest by the men in their work, and all those who handled Barber-Greene machines realized that it was not the machine which produced the whole of the finished product, and unless there was that interest and skill on the part of the foreman and the men working with him the desired result would not be obtained.

The Chairman said he had been dismayed to hear the Author, and at least one other speaker, say that they wanted the Barber-Greene fitted with a lower gear so that it could carry on continuously and yet go at a slower rate. He thought that that was a retrograde statement for anyone to make. To-day it was necessary to gear up production to suit the machine, and a job was not being run properly unless the machine was operating at its maximum. The line which should be adopted was not to make 200 tons a day for the Barber-Greene, but 1,000 tons, as the Americans did, mixtures and outputs being designed so that the machine could be used economically. That was an important point, and he would hate to think that anyone seriously wanted to cut down the gearing of the Barber-Greene, so that less material might be laid while keeping the machine working all day.

The Author, in reply, dealing with the Chairman's point about the speed of the Barber-Greene machine, said he did not suggest that they should not aim at the maximum output, because he thought that they should, and he agreed with the Chairman that a lower gear would be a retrograde step if continuous working of the machine could be ensured by increasing the rate of supply of asphalt. The fact was, however, that in East Anglia, which was a fairly remote area, the gravel pits were generally some distance apart, and difficulty had been found in getting sufficient output to match the laying capacity of the Barber-Greene. The conditions had to be accepted as they were found. It was not possible to get enough material to lay 52 tons an hour, and, until it was, the Author must maintain his point that, as regrettable as it may seem, a lower-geared Barber-Greene was necessary for the best workmanship.

Mr Bowdler mentioned fusion with the old road surface when applying the gravel asphalt for re-shaping purposes. To illustrate the value of that, the Author could quote figures to show the financial saving which that property of fusion might bring about because it enabled scarifying of the old road crust to be eliminated. He had estimates of reconstruction work with coated macadam and with gravel asphalt, and those estimates relates to the same lengths of road. The cost for coated macadam was £20,997 and for the same work in gravel asphalt the cost was £16,793. With regard to what Mr Bowdler had said, in the estimate for the coated macadam work no less than £1,580 was to be spent on scarifying, whereas with the gravel asphalt work on £175 was for scarifying, so that that bore out strongly his remark.

Mr Bowdler had claimed that the cracks in the asphalt over joints in a concrete foundation tended to heal in hot weather. The Author was not satisfied that the asphalt was in fact sufficiently ductile for the crack to close and suggested that any apparent diminuation of the crack during hot weather was more likely to be the result of normal expansion due to temperature changes. In the same way pre-coated chippings standing slightly proud of the surface were not pushed further in under the influence of traffic during hot weather. That point was of some importance because some Surveyors were under the mistaken impression that under summer temperature conditions in Great Britain the asphalt was subject to appreciable movement. That was not so, in the Author's opinion.

Mr Husband had asked for some comparative prices for gravel asphalt work and the local hoggin work which had been done in East Anglia. The price of the hoggin, consolidated, was 10*d.* per sq. yd. per inch of thickness, and the price of the hoggin, stabilized with 10 per cent of cement, was 1*s.* 6*d.* per inch of thickness. The price for the base-course asphalt on the work was 2*s.* 6*d.* per sq. yd. per inch, but the Author did not think that it was possible in such a case to regard the base-course asphalt as a substitute for the hoggin or stabilized hoggin. The base-course asphalt was being placed in a thin layer on top of the stabilized hoggin. The highway authority was getting a very cheap base in the hoggin and stabilized hoggin and a fairly cheap surface in the gravel asphalt, so that they got inexpensively a big thickness of road-stone on a soil of low bearing capacity.

The actual thicknesses and prices per sq. yd. were :—

Inches		£	*s.*	*d.*
6	consolidated thickness of hoggin		5	0
6	consolidated thickness of hoggin stabilized with 10 per cent cement		9	0
4½	consolidated thickness of base-course asphalt .		11	3
1½	consolidated thickness of wearing-course asphalt		5	3
18	inches total thickness. Total cost .	£1	10	6 per sq. yd.

The Author did not think that base-course asphalt would normally be as inexpensive as stabilized soil, for which it should not be regarded as a substitute. It might, however, often be useful for surfacing a stabilized-soil base; and there were frequent occasions when it could be used for work for which the latter would not be suitable.

What Mr Tebbutt said about the asphalt work laid so close to the Institution building in Birdcage Walk was particularly interesting. He did not say that it was gravel asphalt, but presumably he intended that to be understood. The information which he gave, and the fact that the work had been done as long ago as 1934, gave one confidence in the life which might reasonably be expected from the work which was being done on the much more lightly trafficked roads of East Anglia.

The Chairman and Mr Tebbutt had both advocated a high stone content, and the Author agreed that the practicability of that required examination. The stipulation of 35 per cent stone content had been dictated by the prime need to make rapid progress with the work. He thought that there was scope for trial, and perhaps experiment if it had not got beyond the experimental stage, with much higher stone contents. He thought, however, that if high stone contents were used, the stone might have to be mixed, because the gravel by itself might not give the best non-skidding results. The gravel might have to be mixed in equal proportions with granite or limestone, but a few trials for purposes of comparison would have to be made.

Mr Tebbutt had rightly emphasized the influence which the cost of the pre-coated granite chippings had upon the cost of the wearing course. The present cost of the chippings was about 6½*d.* per square yard.

Increasing the stone content to 60 per cent and using equal proportions of graded gravel and granite, with the omission of any pre-coated chippings, was likely to add about 3½*d.* per square yard to the cost of the wearing course. There might, however, be some reduction in the cost of base-course material because men normally employed for spreading pre-coated chippings would not have to be on the job at all whilst the base course was being laid, so that there would presumably be a small saving in labour cost.

In reply to Dr Lee, the Author stated that variations of an exploratory nature had not been introduced into the work in the early stages because the over-riding consideration had been rapid progress and there had not been time to think of experiments. Later on, as experience was gained and different conditions encountered, a number of variations to the specification were made, as described on pp. 179 and 180.

The effect of using different gravels had not been considered. For economic reasons the nearest suitable local gravel had always been used and all the gravels were of the general type described in the Paper. Any other gravels would have had to be imported and the whole object of using local material would then have collapsed.

The bitumen content might have tended to be slightly higher than was necessary, but there again no experiments had been made. There was every indication that the bitumen content specified was satisfactory and there had therefore been no practical reasons for seeking a variation.

The Author agreed with Dr Lee that further comparison between the gravel asphalt and the gravel macadam described in the Paper was needed. On the trunk roads the gravel macadam had been laid for direct comparison with the asphalt. The present indications were that the gravel asphalt was far superior and it was thought unlikely that the gravel macadam would last for the 10-year period suggested by Dr Lee. The Author did agree, however, that more gravel macadam might usefully be laid, under carefully controlled conditions.

Several speakers had emphasized the importance of control, and the Author thoroughly agreed. Dr Lee had criticized the analytical control, and the Author agreed that more extensive control had been desired. The reason, as explained in the Paper, was that it had been found rather costly. While the cost of having the analyses made was perhaps small compared with the total cost of the work being done, nevertheless the bills were fairly high and the Author had been led to wonder whether some saving in cost could not have been made on that item. That was why the system, described in the Paper, had been devised of testing the material for 3 days running, and if at the end of the 3 days the results received from the analyst were satisfactory it had been tested less frequently, reliance being placed upon the discretion of the Clerk of Works, who was experienced and able to form some opinion of the quality of the material by inspection. When he felt the need for another analysis it was made. It would have been much better, however, if more frequent analyses had been made continuously.

That raised the question of a local laboratory, which the highway authority had never possessed. There was a first-class case for the establishment of a local laboratory, perhaps to be used jointly by the Ministry and different highway authorities, or under the control of individual highway authorities. That was most urgently required if the best results were to be obtained.

The Author would like to see the contractor and the local authority co-operating in making the analyses, as had been mentioned in the discussion. He did not see why the contractors and the local authorities should not make a joint analysis, and agree what the content of the asphalt was, instead of the local authority sending the material away to some independent analyst and getting a result with which the contractor did not agree. The contractor then produced evidence from his own laboratory, which was probably just as well equipped and staffed as that of the independent analyst, and the highway authority was in a difficulty about which to accept, knowing that the contractor was probably right.

The Author replied to Dr Lee's observations about the use of continuous plant by remarking that a continuous plant had been used by one Contractor for surfacing 6 or 7 miles of trunk road under conditions where the control exercised was not completely satisfactory. The asphalt laid by that Contractor, who incidentally had not been very experienced, was not generally equal to that which had been made in batch-mixers. There had been some segregation of the stone and in places the material appeared richer than was necessary. There was at present, however, no apparent need to question the durability of the surface, which was laid about 5 years ago.

The Author agreed that it would be interesting to lay some more work with a continuous mixer under the right circumstances and under controlled conditions. Under ordinary work-a-day conditions, however, there was no temptation to depart from the present practice of using batch-mixers.

The Author thoroughly agreed with what Dr Millard had said about the job being a reasonable size. In the Paper the Author mentioned 65,000 square yards, 5 miles of road, as being what he called the optimum, because when letting contracts efforts were made to keep them to about that size. It was generally accepted as a desirable minimum, and one way in which that might be achieved was by authorities combining. That might sound an unusual suggestion, but it had been done in 1951 in Cambridge, where the County Council and the Borough Council had had to surface a continuous length of trunk road. It had seemed unwise to divide the work, and it was arranged that, although two contracts were let, only one contractor was employed, and the two sections of work were done together. A joint advertisement had been issued, and the firms who tendered knew that if they got the one job they would get the other. By that means continuity had been obtained throughout the whole length of road, and the job had been of a more worthwhile size, which meant that the contractor was better able to employ good supervision and adequate plant. The result fully justified any extra trouble which had to be taken when the work was being advertised.

The Author had been asked how the enriched base-course material was standing up. None of it was very old, but it looked very promising. The one-year free maintenance of the enriched base-course material was not a measure of its durability, but represented what was asked for by the highway authority when inviting tenders. They asked for only one year's free maintenance, and the contractors gave it, but had 3 years been asked for it would doubtless have been given, so that the one-year period should not be taken as a measure of the durability.

Dr Millard's suggestions about increasing the supply of asphalt to the Barber-Greene finisher were not generally practical under the conditions which had obtained in East Anglia. The Author agreed, however, that they represented an ideal which should never be forgotten.

The Author thanked Mr Lacey for drawing attention to the human element which he agreed was of first importance. Some contractors, partly because they were fortunate in having the services of out-standing foremen and other employees, had produced better work than others.

There were other contractors who had produced unsatisfactory work. The Author considered that, in the laying of hot-rolled asphalt, a strong case could be made out for restricting tenders to firms of proven competence and reliability so that the employment of unsatisfactory contractors would not be likely to occur. In the programme described in the Paper, three trunk road contracts at least had had to be let to firms having insufficient experience but who had submitted the lowest tenders. Those firms were not encouraged to tender for future work, but it would have been very much better if they had never been employed in the first place and the Author considered that for hot-rolled asphalt work a better return for public expenditure was likely to be ensured if contractors wishing to tender had first to satisfy the Highway Authority that they were competent.

In replying to Mr Lacey's question about the renovation of worn-out asphalt, the Author thought that the first maintenance work required might be surface dressing. Anything more comprehensive than that would involve either the laying of a new surface on top of the old or alternatively removing the old wearing course before laying the new surface.

In reply to Mr Bradbeer, the Author observed that the maximum thickness of base-course asphalt used in the programme under discussion for the purpose of superelevating bends had been about 18 inches.

The results given in Table 1 of readings on gravel-asphalt surfaces all applied to the programme under discussion. The other three surfaces in the Table on which results had been obtained were laid at the same time that the gravel-asphalt programme was in progress.

Paper No. 5833

"Marine and River Surveying with a Circle Chart"

by

John Francis Causton Swansbourne, A.M.I.C.E.

(Ordered by the Council to be published with written discussion) †

SYNOPSIS

The Author describes a method which he has adopted to simplify certain aspects of marine surveying for civil engineers.

The method involves the use of an easily constructed circle chart based on normal geometric principles to replace the cumbrous use of station-pointers. The Paper shows how the use of the chart eliminates technical assistance to a minimum, speeds up the rate of plotting and generally assists in the navigation of the vessel, an operation so alien to a civil engineer that it is desirable for him to have the simplest tools for the purpose. The Author describes several instances in which the system has been used. Three figures depicting the geometric build-up of the circles and two illustrating the way in which they have been used are included, together with a standard Circle Sheet that could be applied to many instances where surveys have to be carried out.

NOTATION

B denotes base line.
B_L ,, left-hand base line.
B_R ,, right-hand base line.
C ,, centre of circle.
H ,, perpendicular, centre to chord.
R ,, radius of circle.
S ,, point on circle.
W ,, beacon point.
Y ,, ,, ,,
Z ,, ,, ,,
A ,, ,, ,,
θ ,, angle subtended at circumference.

A civil engineer is often called upon to execute surveys in rivers, ports, and harbours, but less frequently to conduct them in connexion with coastal development, this being an operation which is generally carried out under the auspices of the Admiralty or a firm of marine surveyors.

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† Correspondence on this Paper should be received at the Institution by the 1st October, 1952, and will be published in Part II of the Proceedings. Contributions should be limited to about 600 words.—Sec. I.C.E.

Occasions occur when it is neither possible, because of time or distance, nor desirable, because of economy, to request the service of either of these bodies of surveyors and the work of mapping the sea-bottom over an extensive area of seacoast is left in the hands of the civil engineer himself.

Provided that he is thoroughly confident in his ability, not only to locate his soundings, but to direct or navigate the vessel along the course required, it is advantageous that he should be entrusted with the responsibility for conducting the work as he can then alter the extent of the survey and necessary degree of detail during the operation afloat. He is in a position to obtain the necessary information in the shortest time since he alone knows the implications of the results. An independent body of surveyors acting on his behalf may spend too much time in obtaining too much detail at high cost or may obtain insufficient information and be required to conduct a second series of trips.

It must be stressed, however, that the engineer should not compromise his relations with the skipper by indecision or hesitation and he must arm himself with a simple and speedy method of locating the ship, of immediately plotting its course and direction at any moment, and of obtaining a rapid picture of the survey as it proceeds.

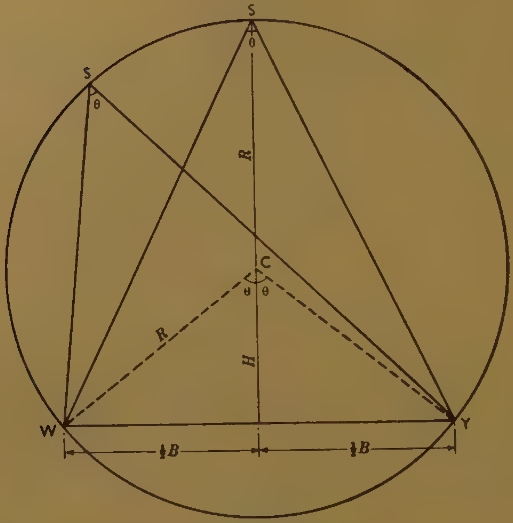
Navigation is simplified, location and orientation is fixed by immediate reference to a chart, and qualified assistance is limited to a minimum. In accordance with the usual procedure in marine surveying, the party is controlled from one point instead of the three which are unavoidable when employing any system of intersecting theodolites. Moreover, the tedious use of station-pointers is avoided and many advantages are described which enable the engineer to conduct a survey with speed and efficiency.

The method is based on the use of a navigation and plotting chart consisting of a system of circles, and involves the well-known geometric property that the angle subtended by a chord at the circumference of a circle is half the angle subtended at the centre. *Fig. 1* shows how the point S describes a locus of points following the circle WSY for a given sextant angle of θ degrees.

If a fixed chord, or base line is selected and the radii R1, R2, R3, and distances XC1, XC2, XC3 are calculated for various sextant angles $\theta 1$, $\theta 2$, and $\theta 3$, then circles, or loci of points corresponding to these values of θ can be drawn as indicated in *Fig. 2*.

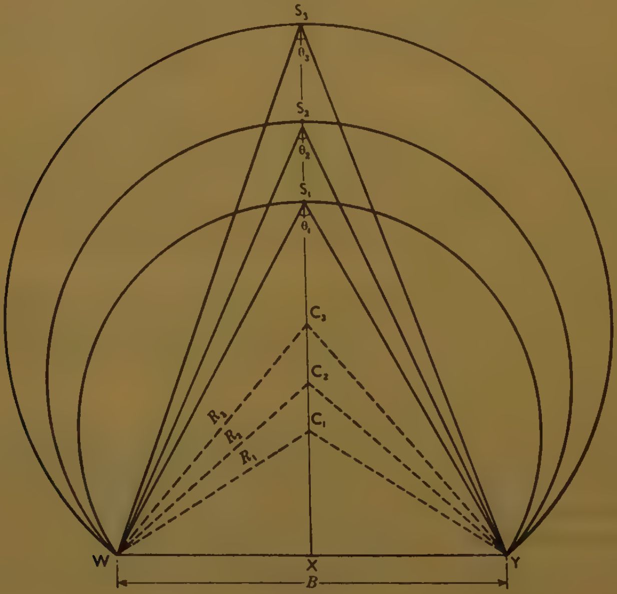
Two sets of circles are required to fix the position of any point S, and these must be plotted on two separate base lines corresponding to the distances between the survey beacons, *Fig. 3*. The intersection of the circles corresponding to sextant angles θ_L and θ_R on base lines B_L and B_R locates graphically the position of the vessel or sounding. Intermediate values of θ must be interpolated. In common with all sextant survey work, care must be taken over the selection of beacons or location of stations in order that a " good fix " is obtained over the important area of the survey.

Fig. 1



Locus of Points Described by a Particular Sextant Angle

Fig. 2

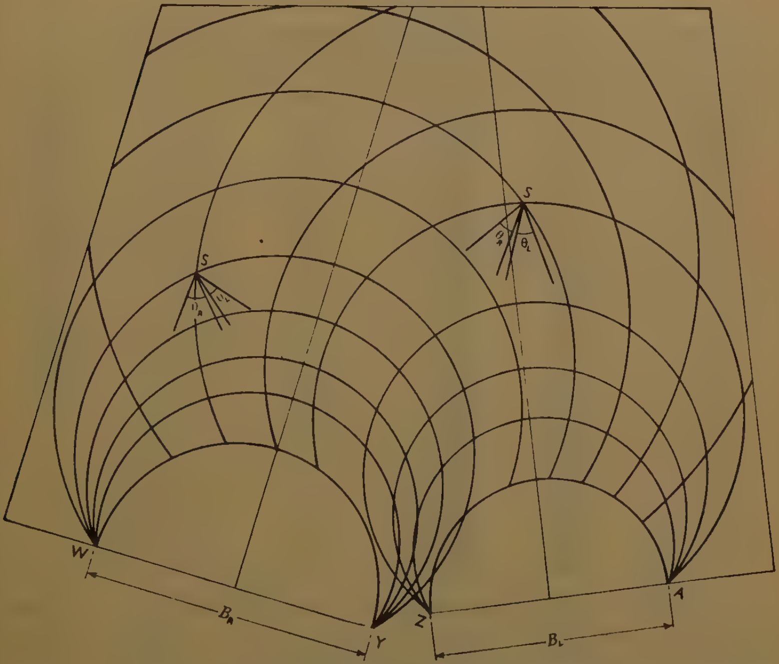


Loci of Points Corresponding to Three Different Sextant Angles, θ_1, θ_2, θ_3

Poor selection of stations will cause certain circles to become concyclic and failure to fix the point S will result.

The system lends itself to a considerable degree of standardization. Except in built-up areas it is usually possible to erect three beacons on a straight line at preconsidered and equi-distant intervals. For such instances the Author has drawn up a standard negative sheet (*Fig. 4*) to occupy the greater part of a double elephant drawing board, from which any number of prints can be taken. Consistent with the adoption of a reasonable scale, the survey would usually be arranged so that it fits on the sheet, and the length of the base lines to be set on land would be calculated to suit.

Fig. 3



Location of Vessel by Intersection of Circles Corresponding to Sextant Readings on Bases B_L and B_R

In order to construct the standard sheet it is convenient to adopt a base line 5 inches long which is equivalent to 200 feet on a scale of 1 inch to 40 feet (see *Fig. 1*).

Then $\frac{B}{2} = 100$ and $H = XC = 100 \cot \theta$;

therefore $R = 100 \operatorname{cosec} \theta$

H and R may, therefore, be read direct from the trigonometric tables. It will be noticed that any length of base involving an odd number of feet necessitates the use of logarithms as an accuracy of four figures is desirable. Table 1 gives values of H and R for various angles of θ which have been selected to give the most suitable distribution of circles on the sheet. The inclusion of half-degree values is not advised.

TABLE 1

For B = 200, R = 100 cosec θ, H = 100 cot θ					
θ degrees	H	R	θ degrees	H	R
100	17·6	—	26	205·0	228·1
90	0	100·0	25	214·5	236·6
85	8·7	100·4	24	224·6	245·9
80	17·6	101·5	23	235·6	255·9
75	26·8	103·5	22	247·5	266·9
70	36·4	106·4	21	260·5	279·0
65	46·6	110·3	20	274·7	292·4
60	57·7	115·5	19	290·4	307·2
55	70·0	122·1	18	307·8	323·6
50	83·9	130·5	17	327·1	342·0
48	90·0	134·6	16	348·7	362·8
46	96·6	139·0	15	373·2	386·4
44	103·6	144·0	14	401·1	413·4
42	111·1	149·4	13	433·1	444·5
40	119·2	155·6	12	470·5	481·0
38	128·0	162·4	11	514·5	—
36	137·6	170·1	10	567·1	—
34	148·3	178·8	9	631·4	—
32	160·0	188·7	8	711·5	—
30	173·2	200·0	7	814·4	—
29	180·4	206·3	6	951·4	—
28	188·1	213·0	5	1,143·0	—
27	196·3	220·3	4	1,430·0	—

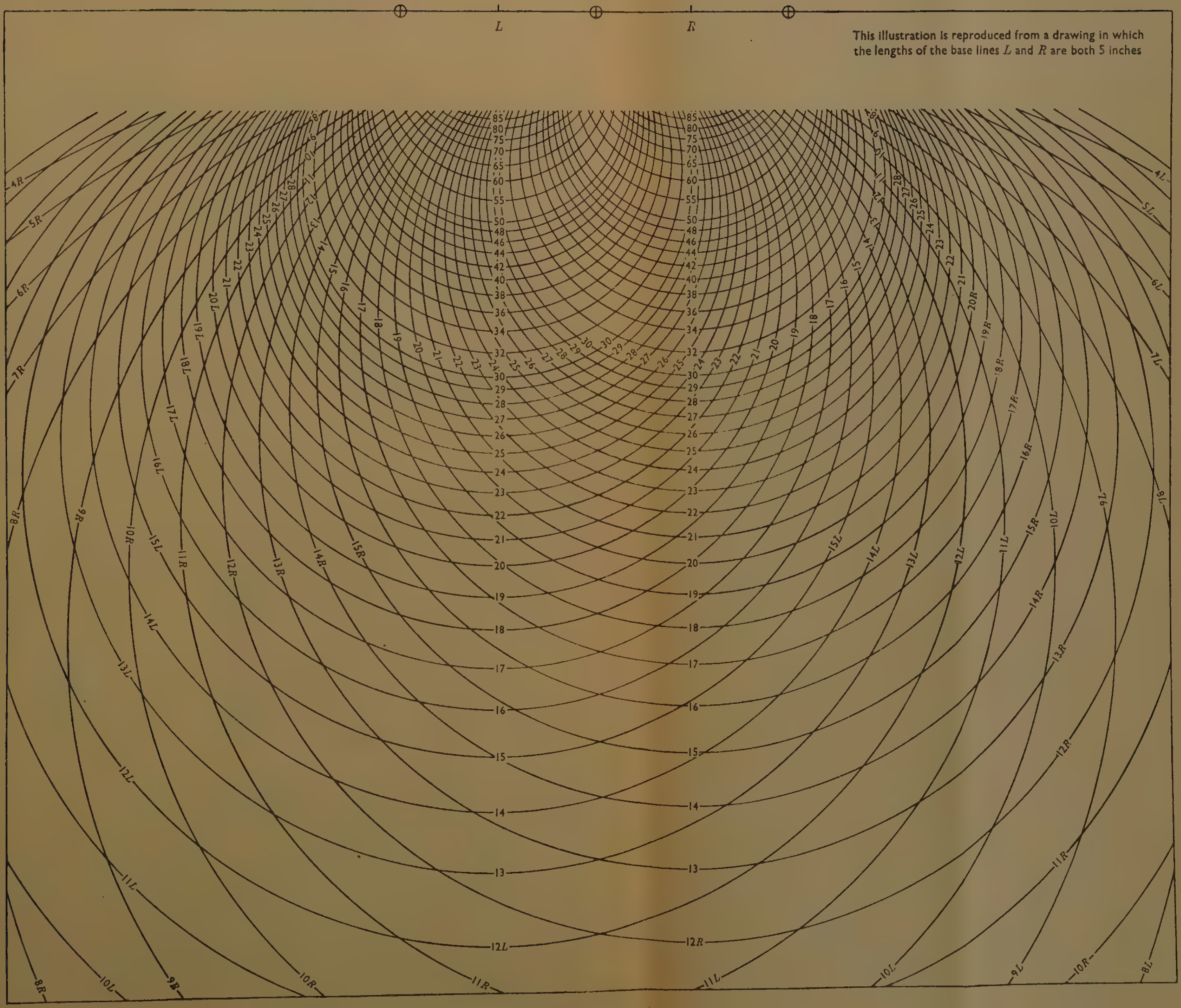
When constructing the circles it is desirable to set off both H and H + R independently from the base line. The compasses are then swept from the apex back towards the base, which results in greater accuracy at the seaward edge of the circle sheet. When θ is less than 12 degrees, the seaward sectors of the circles do not lie on the chart, so that H alone need be calculated and those sectors of the circles that lie on the sheet may be struck off directly from the base stations.

It will also be noticed that for any given angle the value of H + R is equal to the value of H for half that given angle:

that is to say, when $\theta = 30$ degrees

$$H + R = 373{\cdot}2$$

Fig. 4



This illustration is reproduced from a drawing in which the lengths of the base lines *L* and *R* are both 5 inches

CIRCLE CHART

and when $$\theta_1 = \frac{\theta}{2} = 15 \text{ degrees}$$

$$H_1 = 373{\cdot}2$$

This expedites construction of circles and checks calculations where it is necessary to use logarithms.

Care must be taken that the original circle chart and prints are made on good quality paper to minimize shrinkage.

The Author's standard sheet measures 30 inches × 25 inches with each base line 5 inches long. It may be felt that the sheet is just too large in relation to the length of the base lines, causing the degree circles at the edges to be rather far apart. But it is advantageous to have a spacious chart even though the accuracy at the edges may suffer slight limitations. A sextant can quickly be read to ± 1 minute and since, this represents about $\pm\frac{1}{20}$ inch at the edge of the paper, the accuracy is suitable for the boundaries of the main survey.

Table 2 gives the area of this sheet for different scales, from which it will be seen that the engineer can alter the scale to suit his requirements.

TABLE 2

Scale	Area : feet × feet	Length of each base line in feet
1 inch to 20 feet	600 × 500	100
1 inch to 40 feet	1,200 × 1,000	200
1 inch to 50 feet	1,500 × 1,250	250
1 inch to 100 feet	3,000 × 2,500	500
1 inch to 200 feet	6,000 × 5,000	1,000
1/500	1,250 × 1,042	208
1/1,000	2,500 × 2,083	417
1/1,250	3,125 × 2,604	521
1/2,500	6,250 × 5,208	1,042

Where it is desirable to use four landmarks on the shore, forming base lines of unequal length and possessing dissimilar bearings, then a separate set of circles must be calculated for each as in *Fig. 3*. Subject to certain limitations in accuracy such base lines may be scaled off the map, and the circles calculated and plotted by scaling on again. However, if H + R is greater than 4B, and if very great accuracy is required in the main survey then both left- and right-hand base lines B_L and B_R should be measured on the ground; but if the construction and use of the chart is compared with the performance of station-pointers this refinement would appear to be unnecessary.

If a narrow strip along the shore is being surveyed then two or more complete sets of base lines are required with, if feasible, a standard sheet of circles for each one, an example of which is depicted in *Fig. 5* where 2,500 feet by 800 feet of Thames water-front was surveyed with three pairs of base lines and three sheets.

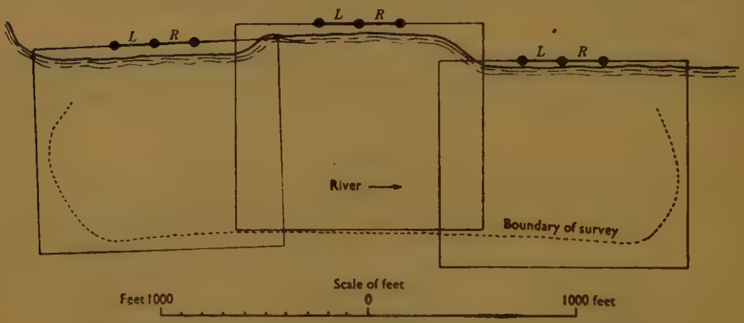
The application of these circle charts having standard equal base lines, or irregular base lines, can with advantage be applied to the following work:

(1) Reconnaissance and construction surveys for harbour development.

(2) Harbour maintenance surveys.

(3) River surveys for jetties, bridges, and maintenance dredging, etc.

Fig. 5

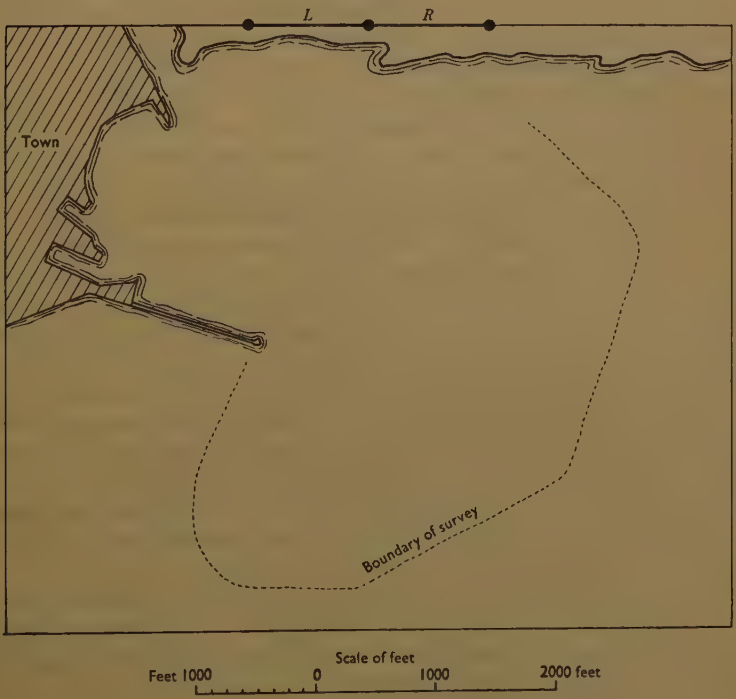


RIVERSIDE SURVEY WITH THREE STANDARD CHARTS

This system has been used for two harbour reconnaissance surveys on the west coast of India. Both surveys were of a similar nature; each covered an area about ¾-mile square and they were plotted to a scale of 1 inch to 200 feet. As there were no suitable landmarks on shore, three beacons were fixed at intervals of 1,000 feet in conjunction with the necessary land survey. *Fig. 6* shows the area covered by one of the surveys. A light tug was borrowed for one day and permitted to drift with the tide at about 1 to 1½ knot. Soundings were taken continuously and logged against the time. The technical complement of the survey party consisted of only two engineers, one of whom was the Author. One engineer was responsible for all booking and reading of the line and the other for reading both sextants and roughly plotting the course. One engineer was able to achieve simultaneous contact with both sextants about once every 2 minutes and keep the course plotted. The intermediate soundings were plotted afterwards in relation to the course and the time. In this manner, an area 3,000 feet long and 2,500 feet wide was covered with about 140 sextant plots together with a continuous run of intermediate soundings where the depth of water ranged between 20 and 60 feet. The following day the inshore strip was plotted from a rowing boat. The survey could have been carried out with the use of station-pointers, but it would have required an additional engineer, or a much longer use of the tug and the whole of the crew.

A further example of a reconnaissance survey along the bank of the Thames was that carried out from a small motorboat with two engineers operating under similar conditions to those described above. Here the survey covered a strip 3,000 feet long and 800 feet wide from the shore (*Fig. 5*) and was generally conducted in courses parallel with the tidal flow. Three sets of double base-lines (L = R = 200 feet) were established beforehand and the survey was plotted to a scale of 1 inch equals 40 feet. Three sets of circle sheets corresponding to each set of beacons were used, the course plotted and the sextant readings logged as the vessel drifted along-shore. Apart from the time spent in setting out the ranging rod beacons, which would otherwise have been spent in setting out the theodolite

Fig. 6



Harbour Survey with One Standard Chart

stations, the survey occupied five river-working hours in the boat with one engineering assistant. During this time 150 fixed soundings were taken.

Later, a detailed survey was carried out by four engineers over a portion of this strip measuring 1,000 feet wide and 600 feet long, one on each sextant, one plotting, and one logging. Sextant readings were taken simultaneously at least every 30 seconds and a corresponding sounding recorded. The area was thoroughly covered in four working hours.

The Author has never used an echo sounding apparatus, and in lieu of the proper sounding chain which has seldom been available, a stranded galvanized iron wire line was preferred to either a rope or a stranded copper line. The rope line stretches and shrinks alternately and presents a greater surface area to the dragging of the water, copper line forms knots and kinks too easily, a tendency which is less apparent in the iron line.

For rapid and reasonably accurate surveying it is preferable to conduct the sounding parallel with the natural drift of the water. The chain or line can then be " humped " along the bottom and readings can be taken continuously. Navigation in this manner is assisted by the fact that the vessel seldom drifts with its hull parallel to the flow of water and a " touch ahead " or a " touch astern " will keep it from straying off the desired course. In this fashion progress is rapid and numerous lines can be completed in a comparatively short time. Although the current is usually parallel with the general direction of the contours, other factors, such as the wind, will modify the course of the vessel and give it a slightly cross-contour drift. Too much cross-current drift is, of course, undesirable as the sounding chain lags behind and must frequently be hauled to the surface. However, in the sea there is usually an interval of slack water in which lines at right angles to the coast can be surveyed as cross checks.

Final plotting may be carried out afterwards in the drawing office by laying tracing paper over the circle sheet and checking against the field book. This method is both extremely accurate and very rapid.

It is obvious that in setting out the beacons for any particular survey, due allowance must be made for the dead area. In a moving boat the sextant becomes unmanageable at 80 degrees as the angular change is so rapid. If, therefore, it is required to survey close to the shore where the base lines are of such a length to enable an area at great distance to be covered, such lines should, if possible, be set inland from the shore to eliminate the dead area. If this is impossible then other arrangements must be made.

If the survey is being carried out in calm water and within 200 feet from the shore then tacheometric location from a central point on land may be preferable. In docks, small harbours, and areas of still water it may be convenient to adopt the time-honoured method of attaching a flagged rope to the shore and semi-technical personnel may then be entrusted with the work.

However, for general purposes, it is considered that when taking account of survey control and subsequent office work, operations conducted with intersecting theodolites are out-weighed by the benefits of total control with sextants on a boat, and furthermore by the use of a circle sheet in lieu of a pair of station pointers.

The Paper is accompanied by 2 sheets of diagrams, from which the Figures in the text have been prepared.

Paper No. 5845

"Construction of a Pre-stressed Concrete Road at Crawley New Town"

by

Arthur John Whyte McIntosh, B.Sc., M.I.C.E.

and

Jack Mercer, M.Sc.(Eng.), A.M.I.C.E.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

The Paper describes a pre-stressed concrete road slab, 404 feet long, which was constructed at Crawley New Town during the summer of 1949. The design of the road was based mainly on the principle that a residual pre-stress of 100 lb. per square inch should remain in the concrete after allowances for subgrade restraint, which was estimated to reduce the stress in the concrete by 100 lb. per square inch in the worst case. Other factors involved in the design are discussed and a comparison made with the normal specification used for concrete roads in the New Town. Freysinnet-type cables and anchorages were used, the 82-foot-long cables forming a diamond pattern at 7-foot-6-inch centres.

The construction of the road and its subsequent behaviour are described in detail. Measurements of movement made by the Road Research Laboratory, which are still continuing, indicate that the expansion of the road does not warrant the construction of a special form of expansion joint. Finally, the cost of the road is shown to be comparable with that of normal construction and conclusions are drawn which may assist future design and construction.

INTRODUCTION

In the summer of 1949 an experimental length of pre-stressed concrete road was constructed as part of the site works of the industrial area of the New Town of Crawley. The work was undertaken in order to investigate the use of pre-stressed concrete in road construction, which had previously received little attention, and this Paper describes the method of design and construction adopted and the subsequent behaviour of the slab.

Before considering the use of this form of construction, an estimate of the probable cost of the road indicated that there should be no appreciable difference between the cost of normal construction and the pre-stressed road. The advantages offered by pre-stressing a road slab are as follows :—

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† Correspondence on this Paper should be received at the Institution by the 1st June, 1952, and will be published in Part II of the Proceedings. Contributions should be limited to about 600 words.—SEC. I.C.E.

(1) The elimination of cracks.
(2) A considerable reduction in the number of expansion joints.
(3) The reduction of maintenance cost as a consequence of (1) and (2).
(4) Reduction in the quantity of materials used, especially cement and steel.

Against these advantages may be offset certain possible difficulties :—

(1) Reinstatement of the road will be difficult where access is required to mains or cables. This problem, however, is conveniently capable of solution in Crawley because all sewers, mains, and cables are laid outside the carriageway, and crossings are concentrated at specific points where a special slab may be constructed over them.
(2) The expensive nature of joints which may have to be designed for comparatively large expansions.

THE DESIGN OF THE PRE-STRESSED ROAD SLAB

The lay-out of the road as constructed is shown in *Fig. 1*, the design being based on discussions between the Authors, the Pre-stressed Concrete Co., Ltd., the Road Research Laboratory, and the County Surveyor of West Sussex. The experimental length was fixed arbitrarily at 404 feet, partly because that length of road fitted conveniently the lay-out of the site and partly because it was felt desirable to construct as long a section of road as possible, bearing in mind the limitations imposed by the indefinite nature of the sub-grade restraint. The remaining aspects of the design are considered in more detail below.

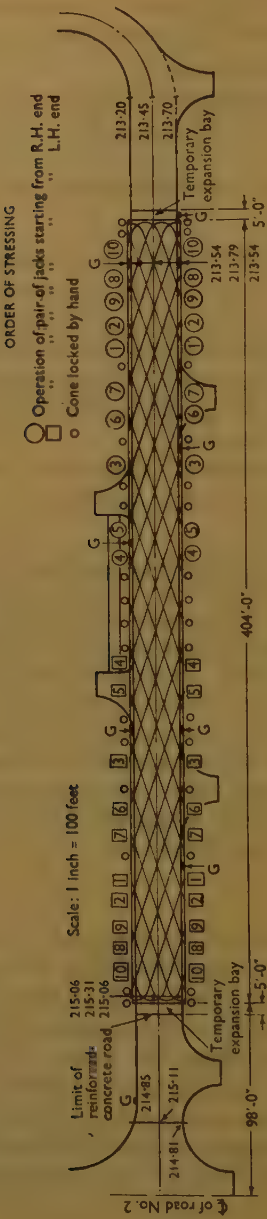
Depth of Slab

Practical considerations made it impossible to reduce the depth of slab to less than 6 inches, since accommodation is required for cables (with sheathing of $1\frac{1}{4}$-inch overall diameter) which cross at different levels in the slab. The cover above and below the sheathing is therefore $1\frac{3}{4}$ inch and it was deemed undesirable to reduce that cover. On the other hand, experience of pre-stressed concrete runway construction at Orly Airport, on a sub-grade of moderate bearing value, had shown that a depth of slab of about 6 inches was sufficient to withstand loads considerably in excess of normal road loading. The depth of slab was therefore fixed at 6 inches.

Pre-stress in Concrete

The estimation of the applied pre-stress was dependent upon the principle that a considerable compressive stress should remain in the

Fig. 1



LAY-OUT OF PRE-STRESSED CONCRETE ROAD

concrete after making allowance for the reduction in stress due to sub-grade restraint. The centre of the slab was assumed to remain fixed and the ends of the slab to move relative to the centre—an assumption which has since proved to be substantially correct. In these conditions, the applied pre-stress in the concrete will be reduced in the worst case by the force required to move half the slab against the frictional resistance of the sub-base. The degree of frictional resistance is difficult to estimate, because there is little available information on the subject. The values of frictional restraint given by Sparkes [1] and Goldbeck [2] cover a wide range and it is evident, from a study of these values, that slight differences in sub-base conditions will cause considerable variation in subgrade restraint. However, information available from the tests carried out by Freyssinet on the Orley runway in 1947 indicated that the coefficient of friction was about 0·22 for the type of construction used (pre-cast concrete blocks on bituminous cloth on sand). The Road Research Laboratory estimated that the coefficient of friction for the proposed sub-base (waterproof paper on clinker with sand screed) should be approximately 0·5. With this latter assumption the loss of pre-stress owing to sub-grade restraint will be 100 lb. per square inch. It was decided, therefore, to provide an initial pre-stress of approximately 200 lb. per square inch, after allowances for losses due to shrinkage and creep, which should provide residual pre-stress of at least 100 lb. per square inch. This is, of course, less than the stresses due to loading and temperature for a slab even of normal length but, when combined with the flexural strength of the concrete, should make cracking under stress unlikely.

Means of Production of Stress

The pre-stress in the concrete was produced by means of Freyssinet-type cables, each consisting of twelve high-tensile-steel wires, 0·2 inch in diameter, grouped around a 16-gauge mild-steel helix. All straight cables were sheathed in 16–18 gauge mild-steel tube, the tube being provided in lengths of 18 feet. Since only one pre-stressed slab was being constructed, it would have been possible to lay all the cables in straight lengths, some of which would have been stressed from the end of the slab. In future work, however, with a series of such slabs this procedure might not be desirable and it was deemed preferable as a guide to future procedure to stress all cables from the sides of the road. The cables which would have emerged from the end of the road were thus curved round with a radius of about 7 feet to emerge from the sides of the slab. In order to provide against uncertain stress conditions which prevailed at the extreme ends of the slab, fabric reinforcement was placed in the top and bottom of the slab for a distance of 6 feet from each end.

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[1] F. N. Sparkes, "Stresses in Concrete Road Slabs." *Structural Engineer*, vol. 17, p. 98 (Feb. 1939).

[2] A. T. Goldbeck, "Friction Tests of Concrete on Various Sub-bases." *Pub. Rds.*, vol. 5 (1924–25), p. 19 (July 1924).

In previous pre-stressed concrete road construction at Luzancy and Esbly, cables were provided at an angle of 45 degrees with the line of the road, to produce equal pre-stress in longitudinal and transverse directions. At Crawley the road width was to be 24 feet with a central longitudinal construction joint and it was felt, therefore, that transverse pre-stress was unimportant. The diamond pattern of cables was used, however, since it made the length of the cable convenient to handle. The general layout of the cables is shown in *Fig. 2*, each cable being at a slope of 1 : 3 with the line of the road at about 7-foot-6-inch centres. The length of each cable after final cutting was 82 feet.

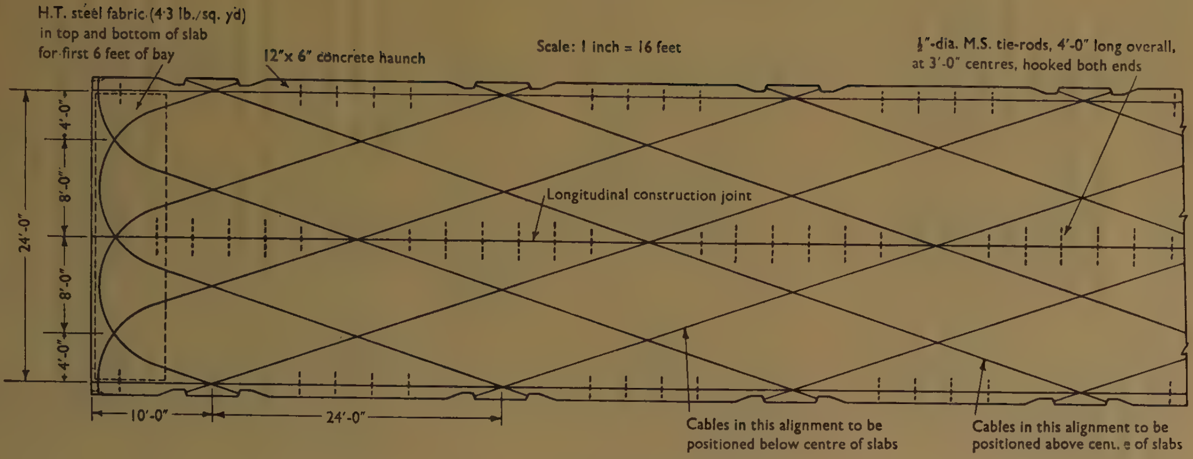
Stress in Cables

For reasons of economy it was decided that a higher stress was permissible in the cables than in normal structural work. Before receipt of the test results on the steel delivered to the site it was anticipated that the 0·1-per-cent proof stress of the wire would be about 95 tons per square inch, in which case it was intended to stress the wire to 87 tons per square inch initially. The stress loss due to shrinkage and creep in the concrete and creep in the steel was estimated at 11 tons per square inch, giving a final tension of 76 tons per square inch. This would provide a final longitudinal pre-stress of 212 lb. per square inch in the concrete. When the test results of the steel were received, however, it was found that the 0·1-per-cent proof stress ranged between 82·3 tons per square inch and 93·8 tons per square inch. And so, when the steel wire was made up into cables, it was impossible to know the precise characteristics of all the different wires in the cable. The extension for each cable, therefore, was so calculated that for the steel with the lower limit of proof stress the extension gave a stress of 80 tons per square inch, and for the steel with the higher limit of proof stress the same extension gave a stress of 83 tons per square inch. Initially, therefore, the average stress was 81·5 tons per square inch and, assuming a final stress of 70·5 tons per square inch in the steel, the load in each cable is 26·6 tons, giving a final longitudinal pre-stress in the concrete of 197 lb. per square inch.

Comparison with Normal Construction

As has already been suggested, the future of the pre-stressed concrete road depends greatly upon the possibility of its construction at a cost comparable with normal construction. At Crawley the subgrade consists of Weald Clay and, after taking a number of soil tests, the depth of normal concrete construction was fixed at 12 inches, comprising a 4-inch lean-concrete sub-base, and an 8-inch main slab as shown in *Fig. 3*. In both cases careful control of the concrete was exercised, weigh-batching of single-sized aggregates being used and the mixes being designed on the basis of Road Research Laboratory Road Note No. 4, to give average crushing

Fig. 2

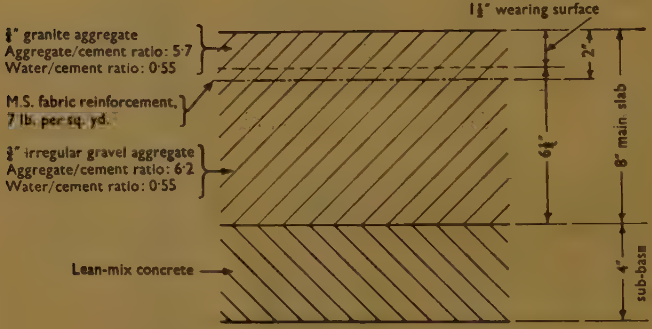

H.T. steel fabric (4·3 lb./sq. yd) in top and bottom of slab for first 6 feet of bay
12"x 6" concrete haunch
Scale: 1 inch = 16 feet
½"-dia. M.S. tie-rods, 4'-0" long overall, at 3'-0" centres, hooked both ends
Longitudinal construction joint
4'-0"
8'-0"
8'-0"
4'-0"
24'-0"
10'-0"
24'-0"
Cables in this alignment to be positioned below centre of slabs
Cables in this alignment to be positioned above centre of slabs

CABLE SPACING

strengths of 4,750 lb. per square inch with minimum strengths of 3,500 lb. per square inch. In practice the average cube crushing strengths were 6,630 lb. per square inch for the gravel aggregate and 5,590 lb. per square inch for the granite aggregate.

In view of these results it was decided that the quality of the concrete was sufficiently high to be used for the pre-stressed road. No amendment was made, therefore, to the specification for the concrete mixes, and the 6-inch depth of slab consisted of 5 inches of gravel aggregate and 1 inch of granite aggregate. It was considered that there was no need to provide a lean-concrete sub-base and a clinker sub-base was adopted for reasons of economy. In order to decrease the coefficient of friction so far as possible, a thin screed of sand was placed over the clinker before laying waterproof paper. A plain butt-joint was adopted for the central construction joint, the face of the concrete being painted with bitumen and the slabs being tied together with ½-inch-diameter tie-bars at 3-foot centres.

Fig. 3



Slab Depths for Normal Construction

The main variation from the normal concrete specification was in the kerb base at the sides of the carriageway. Normally the width of the carriageway is extended by 12 inches on each side in order to provide a base for a 6-inch-by-5-inch pre-cast concrete kerb, which is laid after the conclusion of building operations. In view of the fact that this extension of the road slab would contain the cable anchorages and that the concrete adjacent to the anchorage might be subject to a high local stress, it was decided to construct, at each side of the road, a separate haunch of high-quality concrete in which the female cones for the anchorage would be fixed. With a steel stress of 80 tons per square inch the local stress in the concrete may approach 7,000 lb. per square inch and the concrete mix was therefore designed with a water/cement ratio of 0·4 and an aggregate/cement ratio of 3·9. The haunch was tied into the main slab by ¾-inch-diameter bars and was also reinforced longitudinally by four $\frac{3}{8}$-inch-diameter bars in order to prevent cracking during the process of stressing (*Fig. 4*). It will be seen later that this separate haunch proved to be an unnecessary complication.

CONSTRUCTION

Sub-base

The sub-base of 4 inches of clinker was laid and rolled over the full width of the carriageway at the commencement of the work and was drained by 3-inch tile drains laid on each side of the carriageway and connected to gulleys. Owing to the difficulty of avoiding some disturbance of the surface adjacent to bays being concreted, the placing and rolling of the final layer of sand was not carried out until preparations were almost complete for concreting each particular section of the road.

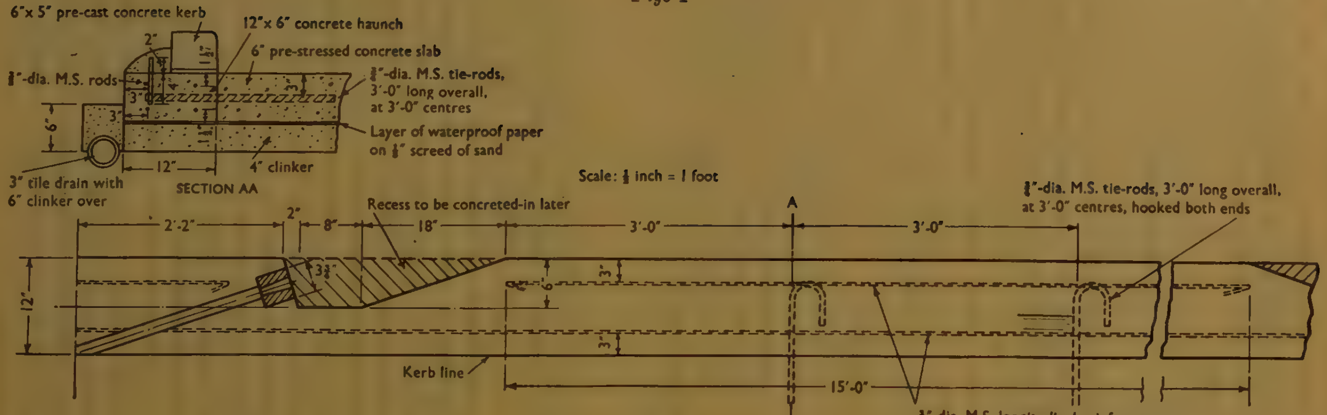
Haunch and Cable Anchorage

Fig. 7 shows the formwork for the haunches with the female anchorage cones in position. After the fixing of the steel reinforcement the concrete was vibrated by a petrol-engined platform vibrator and cube tests at 7 days averaged 5,650 lb. per square inch—a strength which was considered to be satisfactory. The haunches were used as outside formwork to the pre-stressed slab, only a single 6-inch steel form down the centre of the road then being necessary.

Placing of Cables

Following the completion of the haunches, the steel tubes used as sheathing for the cables were placed approximately in position. The end of each tube was opened to form a socket and the end of the next tube forced into it, all joints being sealed with adhesive tape. Although it would have been possible to concrete the slab without the cables inside the ducts it was considered advisable to insert the cables first in order to increase rigidity (*Fig. 6*). Owing to the rigid nature of the sheathed cables, supports at about 6 feet intervals were found to be ample, but it was essential to adjust the cables as concreting proceeded because the placing of concrete was liable to cause slight displacement. In the case of the curved cables, holes were pre-formed in the concrete by means of " Ductube " which was supported by the short length of fabric reinforcement at the ends of the slab (*Fig. 8*). After the completion of the concreting, the cables were pulled through the curved ducts using a woven wire cable grip, a short length of the helix being removed from the cable. Some difficulty was experienced in this procedure. The use of a comparatively rigid form of sheathing was found to be a great advantage during the positioning of the cables and the process of concreting. Without this rigidity, there would undoubtedly have been a tendency for the cable to deflect since the vibrating beam is only $1\frac{3}{4}$ inch above the tube.

Figs 4


6"x 5" pre-cast concrete kerb
12"x 6" concrete haunch
6" pre-stressed concrete slab
⅜"-dia. M.S. rods
⅜"-dia. M.S. tie-rods, 3'-0" long overall, at 3'-0" centres
Layer of waterproof paper on ½" screed of sand
4" clinker
3" tile drain with 6" clinker over
2"
3"
3"
3"
6"
12"
SECTION AA
Recess to be concreted-in later
2'-2"
2"
8"
18"
3'-0"
3'-0"
3½"
3"
6"
3"
12"
A
A
⅜"-dia. M.S. tie-rods, 3'-0" long overall, at 3'-0" centres, hooked both ends
Kerb line
15'-0"
⅜"-dia. M.S. longitudinal reinforcement placed 1½" from top and bottom of haunch
PLAN

Scale: ⅛ inch = 1 foot

SIDE HAUNCH AND ANCHORAGE

Concreting of Main Slab

Following the positioning of the sheathed cables or ducts the concreting of the slab proceeded in accordance with normal practice (*Fig. 9*). The central longitudinal joint enabled the slab to be laid in four operations, the size of each day's work being 200 feet by 12 feet. Across the central transverse construction joint was placed a short length of fabric reinforcement to ensure continuity. The concrete was vibrated using a tamping beam fitted with a curved steel shoe and two petrol-driven vibrators. Cores taken from the normal 8-inch slab had indicated that this equipment was capable of providing good compaction for the full depth of 8 inches. Owing to the length of the slab a number of contraction cracks appeared in the concrete within 24 hours and the behaviour of these cracks will be considered in more detail later.

Stressing of Slab

The stressing of the cables was carried out 3 weeks after the completion of the concreting of the road. The order of stressing proposed originally is shown in *Fig. 1*, and it will be seen that the intention was to apply the stress as evenly as possible over the full length of the road. Four Freyssinet jacks were used, operated in pairs from two pumps, and it was proposed to stress four cables simultaneously, one end of each cable being locked by hand. In the initial stages, however, hand-locking was not found to be completely effective because a few wires slipped relative to the others. A jack was therefore used at each end of a cable, a process which slowed down the stressing programme, since only two cables could be stressed simultaneously. The extension used on the straight cables was $5\frac{9}{16}$ inches and the procedure adopted for stressing was as follows :—

(1) Take up slack on both jacks.
(2) Extend one end by $\frac{9}{16}$ inch.
(3) Extend the other end by 5 inches.
(4) Lock male cone on first end and release jack.
(5) Lock male cone on second end and release jack.

In all cases the extensions were checked against the gauge pressure on the pumps. In the case of the curved cables the extension at each end was made proportional to the distance from the centre of the curve. The checking of the extension of the cable against the gauge pressure indicated that there was no appreciable loss of stress owing to friction in the bends.

In two cases the concrete of the haunch failed during stressing owing to incomplete compaction of the concrete. This was due to the small amount of cover to the female cone—$1\frac{1}{8}$ inch—and it is considered that the cover should be a minimum of 2 inches. In another case, owing to slipping of wires, the jack at one end was released on to the female cone with sufficient force to shatter the concrete surrounding the cone. In all these cases the surround to the female cone was replaced, using high-alumina cement, and stressing took place successfully after 24 hours' delay.

Grouting-up of Cables and Completion of Haunch

On completion of the stressing, the cable wires were cut off 12 inches from the cones and the cable ducts were injected with clean water under pressure. The ducts were then grouted up. Equal volumes of sand, cement, and water had been specified for the purpose, but difficulty was experienced with this mix and neat cement grout was finally used. Following this operation the anchorage-cone recesses in the haunch at the side of the road were concreted up.

Expansion Joints

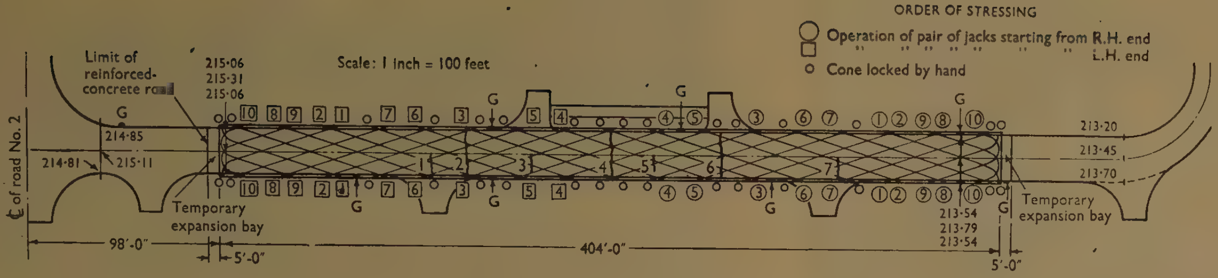
At the commencement of the work it was impossible to calculate accurately the expansion which would take place at the ends of the prestressed road. It was decided, therefore, to place a temporary expansion bay, 5 feet long, at each end of the road. Each bay has a hardcore foundation surfaced with tarmacadam and cold asphalt and incorporates a 2-inch thickness of expansion-jointing material. The actual contractions over a period of 9 months appear to be of the order of 0·5 inch and after confirmation of this during next winter an expansion joint will be designed to fit this movement. It seems probable that an expansion joint of normal form at each end of the bay with a total width of 1–1¼ inch will suffice.

Care was taken to avoid any restraint of the longitudinal movement of the slab at each side of the road, by providing sliding joints at gulleys and points of intersection of kerb and carriageway.

Contraction Cracks in Road Slab

As has been previously mentioned, contraction cracks took place in the concrete soon after placing, the positions of which are indicated in *Fig. 5.* Before stressing, the width of the cracks varied considerably with change of temperature. They were protected during the progress of the work to prevent the ingress of dust and grit and observations were taken on the width at frequent intervals during stressing and from time to time afterwards. It must be emphasized that, owing to the large daily temperature range during August, it is impossible to differentiate accurately between opening and closing of the cracks caused by temperature changes and the stressing of the slab. Nevertheless it was clear that stressing of a cable crossing the crack caused closure and, in the case of crack No. 6, when cables were stressed on both sides of the crack (but not across it), opening took place to the extent of ⅛ inch. However, all those cracks closed up during the process of stressing and, at the completion of stressing, only crack No. 6 was measurable at 0·03 inch, although the remainder were just visible. The protection of the cracks was maintained for several weeks until all were completely closed up. In the case of crack No. 6, a certain amount of tearing of the small aggregate particles took place during the initial cracking and as a result, therefore, the surface of the crack is still plainly visible although the faces are fully in contact. This latter fact is

Fig. 5



POSITIONS OF CONTRACTION CRACKS

proved by the measurements of slab movement made by the Road Research Laboratory and mentioned later. It is evident that in future work an attempt should be made to avoid contraction cracks before stressing and the following procedure would assist :—

(1) Construction in spring or autumn, when the daily temperature range is least.
(2) Use of more construction joints than were used at Crawley, in order to limit cracks to definite points and to prevent tearing of the concrete surface at random cracks.
(3) The use of a limited amount of pre-stress as soon as possible after concreting. This would, of course, involve increased cost.

Measurement of Movement and Strain

Little is known at present about the amount of movement of a road slab of this length and, in order to investigate the problem, measurement of the movement of the slab at seventeen points has been carried out by the Road Research Laboratory. In order to do this, fixed concrete blocks were established at the sides of the carriageway, each block being anchored by short piles taken down to a depth of 8 feet. The gauge used for measuring the movement of the slab is shown in *Fig. 10.* The number of variables which cause movement of the concrete during the early stages is considerable and as a result it has been impossible so far to make any reliable estimate of the subgrade restraint. It seems fairly certain, however, that the magnitude of this restraint is not high and that the original estimate of a coefficient friction of 0·5 was reasonable. It is possible that future measurements, when movement caused by shrinkage of the concrete is much less, will enable an estimate of the subgrade restraint to be made. The measurements show that the maximum inward movement of the ends of the slab, during the winter of 1950–51, relative to the position before application of stress, were as follows :

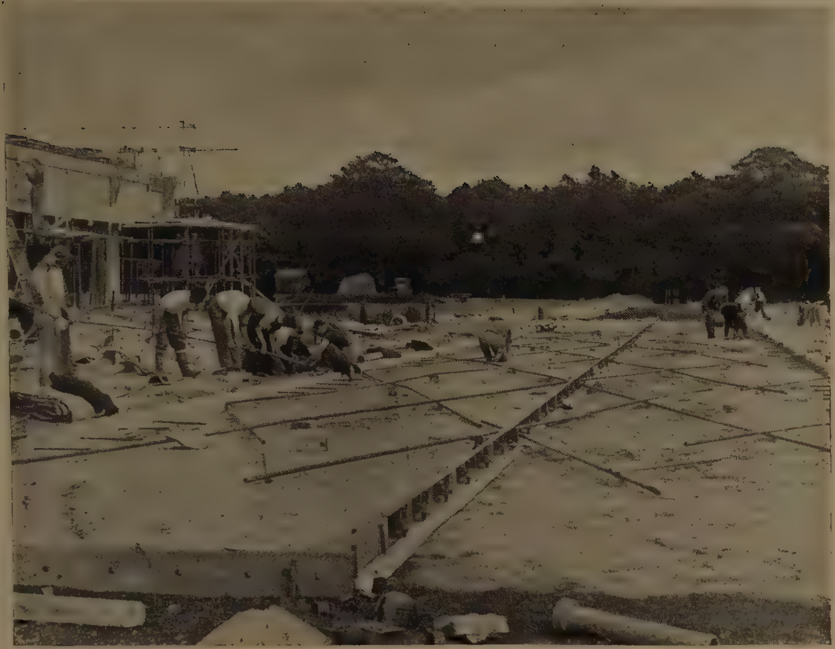
west end, north side : 0·46 inch
west end, south side : 0·56 inch
east end, north side : 0·52 inch
east end, south side : 0·54 inch.

Strain gauges of an experimental type were inserted in the road by the Road Research Laboratory, in an attempt to measure the strain caused by stressing the slab. Measurements were taken during the process of stressing and afterwards but further experimental work will be necessary before confirmation of the results can be obtained.

Cost

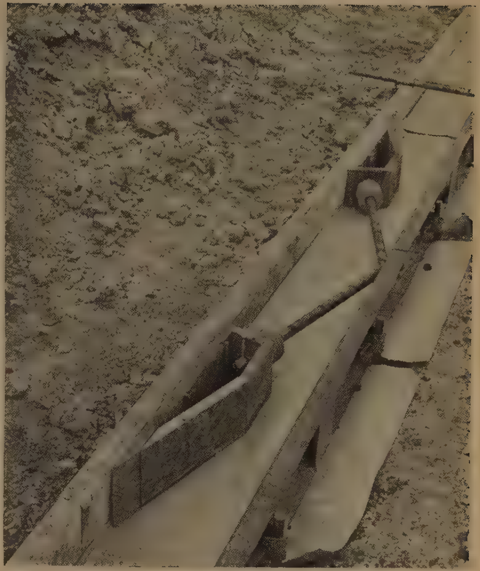
Owing to the experimental nature of the work, payment to the contractor was made on a daywork basis, a procedure which does not give a

Fig. 6



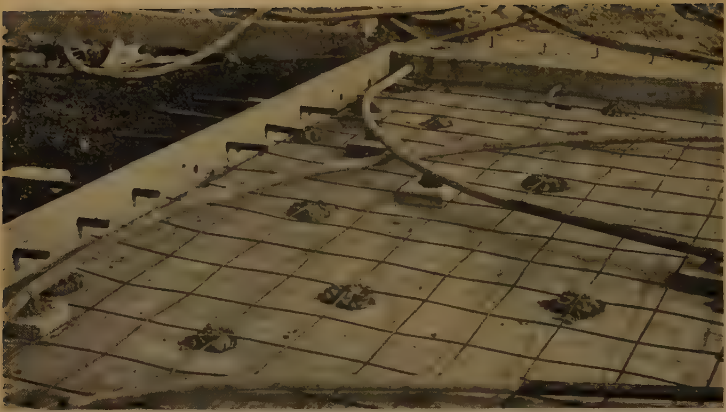
Insertion of Cables into Sheathing

Fig. 7



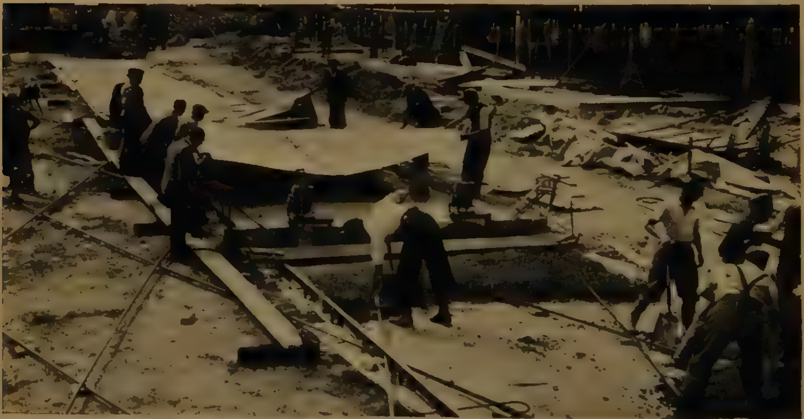
Formwork for Haunch, showing Female Cones in Position

Fig. 8



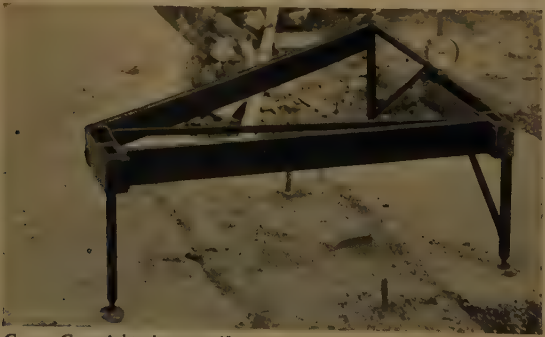
END OF PRE-STRESSED SLAB, SHOWING END OF SHEATHING CONNECTED INTO CURVED "DUCTUBE"

Fig. 9



CONCRETING THE MAIN SLAB

Fig. 10



Crown Copyright photograph]

GAUGE USED BY THE ROAD RESEARCH LABORATORY FOR THE MEASUREMENT OF SLAB MOVEMENT reliable indication of the cost of the road under normal circumstances. However, after the completion of the work, tenders were invited for a similar length of road. The contractor who constructed the first length of road tendered for the further section at a cost slightly less than that of the normal construction which has already been described. Moreover, the average of the four lowest tenders received was only 6 per cent greater than the cost of normal construction. There seems to be little doubt, therefore, that a pre-stressed road could be constructed at a cost comparable with normal construction. This may not be true, of course, in an area where the stability of the subsoil enables a comparatively light form of construction to be used in normal circumstances.

CONCLUSIONS

Whereas further measurements of slab movement and strain may provide information of value, it is already possible to draw certain conclusions from the construction of the road :—

(1) Although the subgrade restraint appears to be of the same order as that originally assumed, it seems logical that the initial frictional restraint is greater than that after a certain amount of movement has taken place. This effect has been mentioned by Sparkes [1] and is borne out by the delayed action of the applied stress in closing the contraction cracks. In carrying out a further section of road of similar length on the same sub-base, it is felt that the stress could be increased with advantage to between 250 and 300 lb. per square inch.

(2) Although the diagonal cables were conveniently short to handle, considerable economies would be effected by using longitudinal cables ; not only is the total length of cable reduced but the number of anchorages and stressing operations would be reduced to a fraction of those used at Crawley.

(3) Some means of control of contraction cracks is necessary. This has been discussed previously but the most practical method would involve the construction of the road in bays with construction joints at 50-foot intervals. These joints would be difficult to detect after stressing but would prevent random cracks.

(4) The side haunch used at Crawley was an unnecessary complication and could be eliminated with advantage. If Freyssinet cones be used, however, the cover to the cones should be at least 2 inches at the edges of the slab.

(5) The expansion of a pre-stressed slab, 400 feet long, is not sufficient to warrant the construction of complicated expansion joints.

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[1] See footnote 1, p. 223.

ACKNOWLEDGEMENTS

The Authors wish to acknowledge the assistance given by the County Surveyor of West Sussex, Mr A. Floyd, C.B.E., B.Sc., M.I.C.E., and by Mr J. C. Howard, B.Sc.(Eng.), A.M.I.C.E., of his staff; by Mr F. N. Sparkes, M.Sc., M.I.C.E. and Mr W. E. I. Armstrong, M.Eng., A.M.I.C.E., of the Road Research Laboratory; by Mr A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., of the Pre-stressed Concrete Company Ltd; by the staff of the contractors, Messrs Willment Bros. Ltd, of Twickenham; and by Mr M. Milne, B.Sc., A.M.I.C.E., and Mr G. Cramp of the staff of the Chief Engineer, Crawley Development Corporation.

The Paper is accompanied by five photographs and two sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Paper No. 5813

"Soil Structure and the Road Engineer"

by

Francis Lesley Digby Wooltorton, B.Sc.(Eng.), M.I.C.E.

(Ordered by Council to be published in abstract form) †

THE stability of a soil used for road foundations depends upon the stability of its micro-structure, which, in turn, is controlled by those site conditions which govern restraint and movements of moisture and hence the variation of moisture content.

Road design methods, based on correlationships and/or testing methods involving empirical considerations and assumptions concerning the final subgrade moisture-content respectively, are suitable in certain areas. The international road engineer, however, must know whether they may be economically applied anywhere—particularly in undeveloped areas whose climates differ essentially from those where the methods were developed.

The research engineer suspects that the subgrade moisture-content may decrease below the construction value in humid regions and may increase in arid regions, but he does not know to what extent or under what conditions this variation need be considered.

The Paper analyses the factors contributing to the soil's resistance to the destructive forces arising during absorption. It includes a review of climatic effects on structure formation and upon the stability of the framework within naturally or artificially compacted soil, and studies of volume changes.

Soil will absorb moisture according to the energy available for absorption, that is to say, surface energy (clay type) and any applicable capillary energy (compaction, entrapped air, effective pore diameter, surface tension forces, and size and shape of the soil mass). Conditions may be such that only a fraction of the capillary energy is effective. The picture is then modified by evaporation, external restraint (deterring air escape and restricting swelling by absorbing some of the energy), structure, and the waterproofing action of certain dehydrated colloids.

Determinations of *pF*-values give little assistance in understanding what may happen during the soil's yearly life-cycle. The oedemeter test may give a result which is never applicable to desiccated soils.

Kersten[1] analysed thousands of subgrade moisture-contents in an endeavour to correlate the values with values of plastic limit, optimum moisture content, and calculated saturation, but there are objections to these methods. For the saturation correlationship, he obtained an average value of 73 per cent with 15 per cent of the samples exceeding 90 per cent saturation.

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† The full MS. and illustrations may be seen in the Institution Library.

[1] The references are given on p. 246, *post*.

The "field moisture equivalent" represents the moisture which a powdered soil, having lost its structure but not necessarily all its moisture-resisting properties, will absorb under its energy conditions. It is a simple test representing an energy state which *must not be altered* by applying external energy.

A properly designed road is waterproof and permits non-capillary void drainage.

Since non-capillary voids will be less in a compacted soil than in a sample used for the F.M.E. test and since subgrade material is continuous and restrained, it is reasonable to assume that the absorption capacity of a covered subgrade is less than the F.M.E.

An examination of subgrade data indicates that the central area takes up or dries out to a moisture content represented by approximately 75 per cent of the field moisture equivalent.

For other energy conditions, the correlationship varies. Thus all the small flat pats used by Porter[2] (from which air could easily escape) absorbed moisture up to complete saturation when the low density samples absorbed moisture slightly in excess of the F.M.E.

Fig. 1 illustrates determinations on an uncovered partly compacted silty subgrade from which air was free to escape. For these conditions, the moisture content approached 87 per cent of the F.M.E. after prolonged rain.

Fig. 2 shows some relationships between the maximum absorption and F.M.E. for various testing conditions.

From the time of emplacement of the coarser particles, accompanied by the deposition of flocculated micelles, a complex network of arched micro-structure has gradually been created and cemented together to form Kubiena[3] braces or Casagrande[4] semi-elastic clay bonds.

Aided by micro-organic activity, there may occur, to a more uniform pattern, colloidal cementation caused by: deflocculation; translocation; flocculation; polar adsorption; apolar adsorption of clay micelles, colloidal organic matter—especially in the presence of multivalent replaceable bases—or its residues; deposited sesquioxides, silicic acid and carbonates; or dehydration.

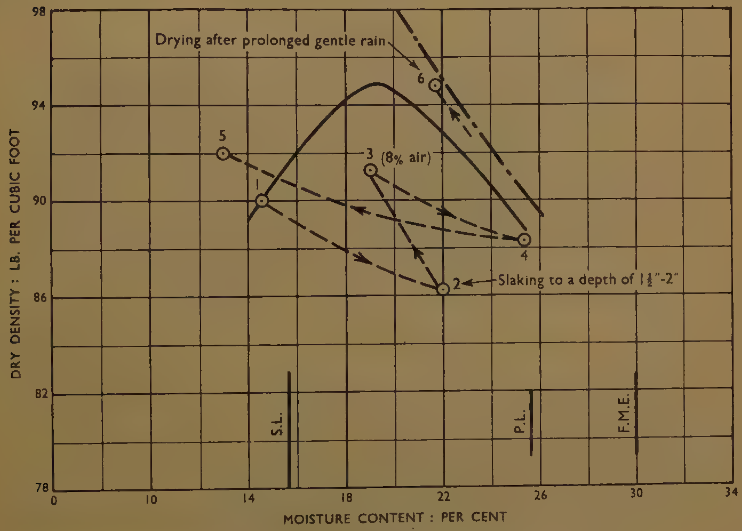
Such structural clay soils tend to be permeable, as lateritic and chernozem soils, though some horizons, such as the B-horizon of some podsolic soils, may be impermeable.

There are also the "structureless" clay soils caused by periodic excessive moisture, or replaceable sodium and some hydroxides and bicarbonates. They are dense and generally impermeable, for example, sodium-type clays (for which $S/R>2$) though not necessarily always so (for example some A-6 solonetz's). Though termed "structureless," they may contain a single-grained micro-structure.

Natural consolidation, in which temperature- and moisture-changes play an important part, strengthens a soil's framework.

The effect of cementation and densification is to produce a material with considerable aggregate or particle-resistance to moisture penetration, though the soil may be permeable. This resistance tends to increase as tropical desiccated soils are approached, though there are notable exceptions.

Fig. 1



Nakuru–Londiani Road (Chainage : 829)

14 July, 1949—30 September, 1949

Specific gravity : 2·33.
Liquid limit : 35.
Plastic limit : 25.
Field moisture equivalent : 30.
Shrinkage limit : 13 (approx.).
(Soil properties variable.)

Disturbing the natural structure decreases shear resistance, and this change may be permanent when only irreversible colloids are present ; but much of the shear resistance may be recoverable, in time, when reversible colloids are included.

Although a recompacted soil may lack shear resistance, cementation may still become effective, if the soil be not overworked, through moisture-resistant coatings, etc. in deterring moisture-absorption. Some lateritic soils—perhaps thixotropic—are exceptions and completely change their properties of permeability, plasticity, and structure when disturbed.

The A- and B-horizons of clay soils existing under similar temperature conditions may be generally expected to have similar water-resisting properties which will tend to improve with increasing rainfall. Those of semi-humid soils will display similar tendencies for a constant rainfall; whilst those of semi-arid soils may be expected to be pseudo-stable by virtue of a combination of densification and the shutting-out-of-water effect, which characterizes the osmotic swelling of sodium-type clays; saline soils and calcium-type clays which are void of organic matter are unstable.

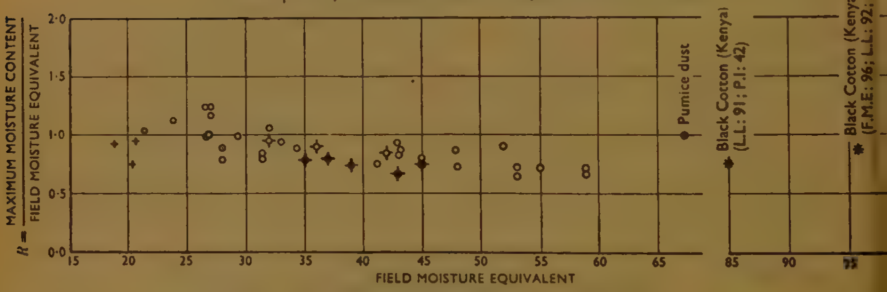
Fig. 2

○ ● denote samples from Nakuru-Molo River road, Kenya. Values determined from optimum moisture content (modified A.A.S.H.O.) plus upward capillary "pick-up" (96 hours). Ends of samples unrestrained and unloaded.

+ denotes values of maximum moisture content (after 96 hours) of undisturbed natural soil at depths of 3 feet (2 samples) and 7 feet (1 sample), over a period of 15 months at Mandalay, Burma. Samples restrained and loaded (L.L. = 67-81; P.I. = 19-24).

✱ denotes pick-up determined after 7 days capillary absorption followed by 30 days total submersion. Absorption appeared to be complete but samples had fractured with some slaking. Sample ends unrestrained and unloaded.

✦ denotes values based on Texas subgrade moisture content survey (reference No. 2, part XI) and available F.M.E.'s. Samples restrained and loaded.



Relationships between the Maximum Moisture Content and the Field Moisture Equivalent for Various Testing Conditions

The tendency to slake increases with :—

(1) Decrease in initial moisture content, that is, as the available surface energy increases.

(2) Increasing rate of moisture supply and absorption when the cohesive clay bonds may be unable to resist particle movements.

(3) Decreasing structural effects.

(4) Decreasing percentage compaction, decreasing restraint, and the application of external energy.

(5) Decreasing ratio of volume of voids per 100 grammes dry soil divided by volume of clay fraction in cubic centimetres.

(6) Increasing temperature.

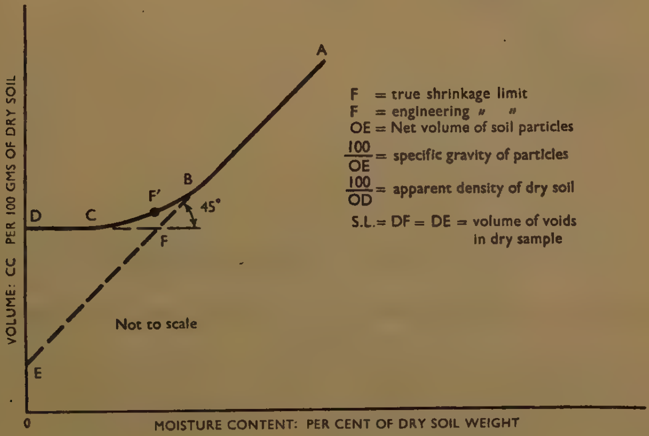
(7) Increasing percentages of electrolytes and free calcium carbonate.
(8) Differential moisture changes.

Condition (5) has been utilized[5,6] for investigating the requirements for a mechanically stable compacted dense graded granular material.

Shrinkage and density-change curve studies for slow moisture changes, involving only internal energy changes, assist in understanding the phenomena involved.

The starting point A for the normal shrinkage test (*Fig. 3*) can only represent a subgrade condition when external energy is applied or slaking occurs.

Fig. 3


A
B
F′
45°
D
C
F
E
0
Not to scale
VOLUME: CC PER 100 GMS OF DRY SOIL
MOISTURE CONTENT: PER CENT OF DRY SOIL WEIGHT
F = true shrinkage limit
F = engineering " "
OE = Net volume of soil particles
100/OE = specific gravity of particles
100/OD = apparent density of dry soil
S.L. = DF = DE = volume of voids in dry sample

TYPICAL SHRINKAGE-TEST CURVE FOR REMOULDED SOIL CONTAINING NO ENTRAPPED AIR

As the moisture falls below B (approximate plastic limit) air enters until, at zero moisture-content, it numerically equals the shrinkage limit. When rewetting from any point above B, the curve retraces the drying line to some point generally below A. On drying to below B, contained air prevents BA being retraced. There will be no subsequent change in the S.L. for the former but there will be in that for the latter. After the first drying, the effect of the initially applied external energy disappears and structure, under the influence of volume-change forces, is in the process of re-formation.

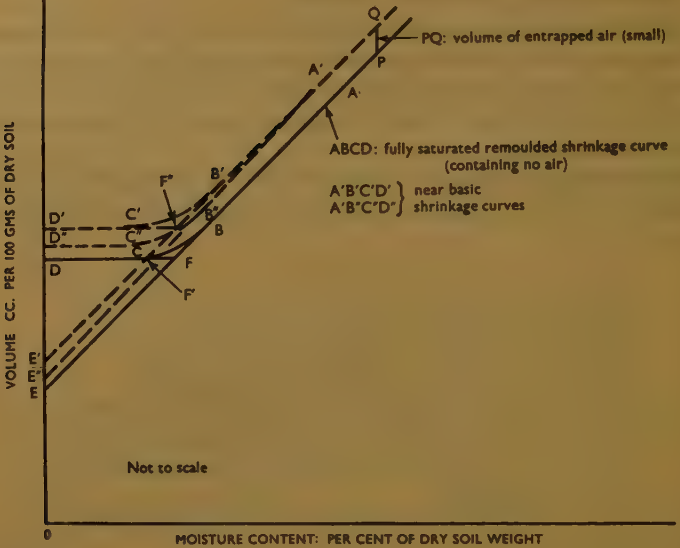
If remoulding does not remove all air, there will be some entrapped air[*]

---

[*] “Entrapped air” is defined as above, or as the small amount persisting after compaction at a moisture content in excess of the optimum. It may be considered to represent, for plate-shaped clays, the air contained in the coarser voids between pairs of approximately plate-shaped micelles forming short capillaries. It probably remains of sensibly constant volume unless external energy be applied to overcome the surface-tension forces of the menisci.

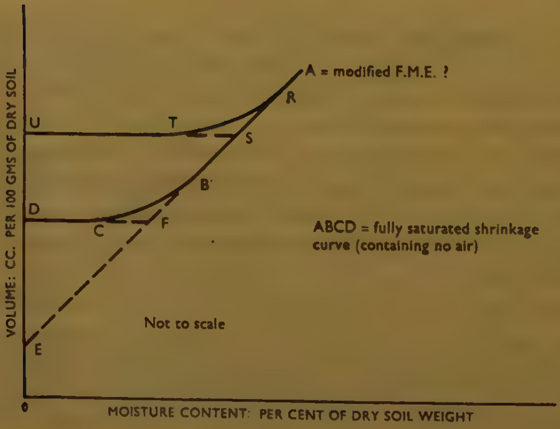
Air in excess of the shrinkage test dry-voids content plus the entrapped air, when a partly compacted or undisturbed sample is dried, is referred to as “expendable” air.

Fig. 4



Shrinkage Curves for Remoulded Soil when some Entrapped Air is Present

Fig. 5

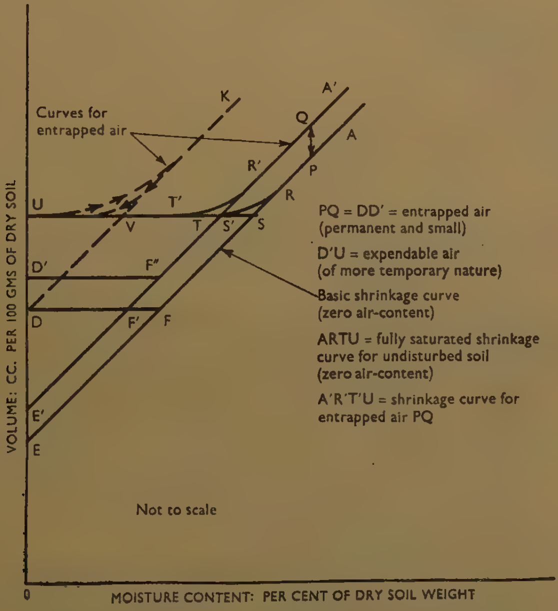


Expected Shrinkage Curve for a Fully Saturated Natural Element and the shrinkage curve lies between A′ B′ C′ D′ and A′ B″ CD, (*Fig. 4*). The variation in slope of the line A′ B″ from 45 degrees has been noted.[7]

The shrinkage curve for a fully saturated natural element would be expected to become ARTU (*Fig. 5*), where the position of A is determined by initial energy conditions. The S.L. is now greater than the normal test-value and the difference is due to structure. Such differences have been recorded.[4]

Undisturbed desiccated soils generally contain entrapped air when "saturated." The expected drying curve would be of the form A′ R′ T′ U, *Fig. 6*. If A′ S′ is at 45 degrees to US, the S.L. equals the fully saturated S.L. minus the volume of entrapped air. It will be noted that the maximum value of entrapped air equals the remoulded S.L. or dry voids-content for the shrinkage test.

Fig. 6

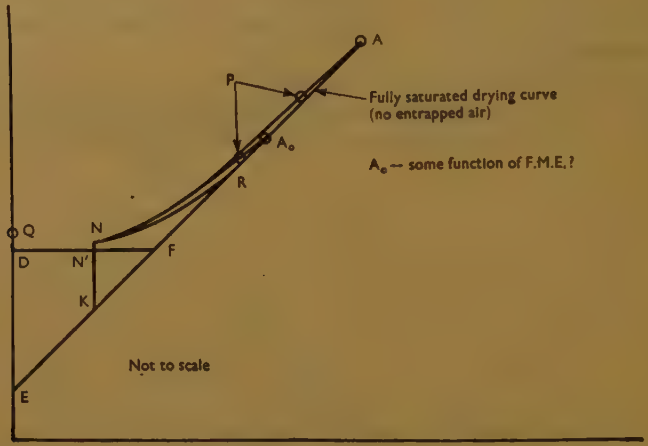


EXPECTED SHRINKAGE CURVES FOR UNDISTURBED DESICCATED SOILS

Hysteresis, which is more characteristic of tropical than temperate soils, occurs during any drying and wetting cycle which permits air to enter. If the cycle includes the first drying from the remoulded state or if, in any cycle, the time permits a change in structure, then the wet end points will differ and the effect is accumulative until equilibrium is reached. There is a change in air voids content and the soil opens out.

Hysteresis may be explained in a number of ways, for example, on a soil-suction basis, or by the differing phenomena operative during drying and wetting phases.

Fig. 7



Drying Curve and Re-wetting Curves for Remoulded Soils when no Air Originally Present

Hysteresis may be operative in the following ways :—

(1) If a fully saturated remoulded sample, represented by A, *Fig. 7*, dries to N below the P.L., air NK enters. There are two possibilities with subsequent absorption :—

(*a*) Slaking occurs with loss of air and structure, the curve becoming approximately NA. During drying, air does not enter until R is reached ; during rewetting, air escapes up to condition A.

(*b*) Slaking does not occur. Moisture enters and some condition A_o is reached. If wetting reaches P only, then air is entrapped and redrying will take place along some curve PQ.

(2) Similarly, a desiccated undisturbed soil drying from M to N″, *Fig. 8*, may rewet to :—

(*a*) A if there is slaking, or to

(*b*) A_o' or A_o'', or even M when there is no slaking.

Under (2)(*a*) the structural condition is destroyed and subsequent drying occurs along a near-saturated line giving a greater apparent density.

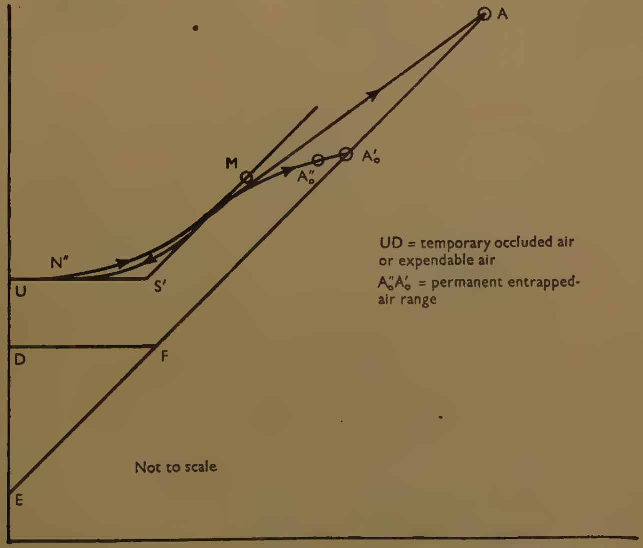
Under (2)(*b*) the structure is maintained and drying occurs along a sigmoid curve.

The lunar curve of *Fig. 7* changes to a sigmoid of *Fig. 8* when the entrapped air exceeds a critical percentage maintained by the structure.

The effective shrinkage limit changes with chemical and physical structure. *Figs 9(a)* and *(b)* show the S.L. decreasing as the compaction density decreases, whilst for undisturbed near-saturated soils it will be greater than the remoulded value ; that is to say higher site S.L.'s are more normal for saturated undisturbed than for remoulded or compacted soils.

It therefore appears that the stability loss of an undisturbed temperate soil, on absorption, is less than for the same soil when disturbed and recompacted to the same density, and that the loss for compacted material decreases as the density increases.

Fig. 8


A
M
A_o'
A_o''
N''
U
S'
D
F
E
UD = temporary occluded air
or expendable air
$A_o''A_o'$ = permanent entrapped-air range
Not to scale

DRYING CURVE AND RE-WETTING CURVES FOR UNDISTURBED DESICCATED SOILS

The undisturbed S.L. of a desiccated soil will be less than that for a near-saturated soil, and it seems likely that the effect of remoulding or compacting will be less than for saturated soils. Chemical structure, involving aeration and dehydration, apparently delays shrinkage and requires greater shrinkage forces to bring the particles into close contact than are required by saturated soils. The destruction of such microstructure appears to result in little depreciation of stability.

For any given amount of compaction there are, after initial drying, constant minimum and maximum densities, which are *dependent* on the amount of compaction. Should slaking occur, repeated drying and wetting cycles eventually result in constant end values which are *independent* of the initial compaction.[2]

Density-change curves, *Figs 9* (*a*) and (*b*), illustrate how the air content affects the amount of volume change when drying or wetting from the compaction moisture content.

Fig. 9 (a)

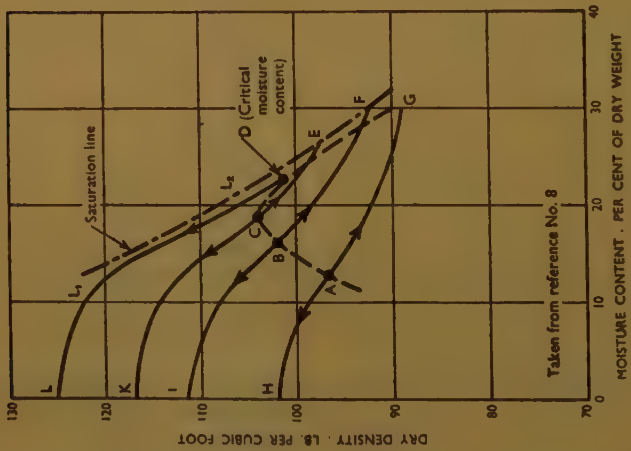
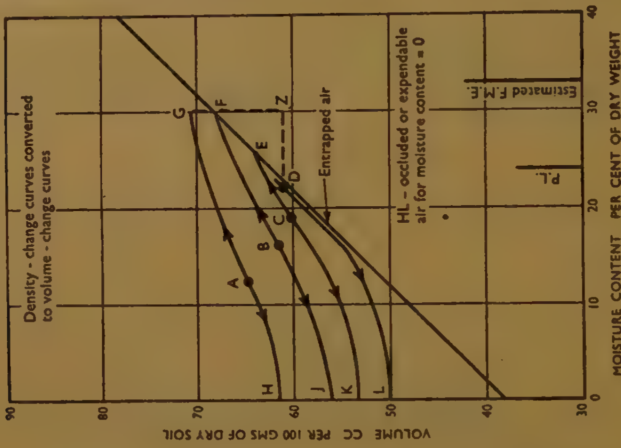


Fig. 9 (b)



DENSITY-CHANGE CURVES FOR SANDY LOAM

Plastic limit : 24. Estimated field moisture equivalent : 33.
Plasticity index : 19. Specific gravity : 2·59 approx.

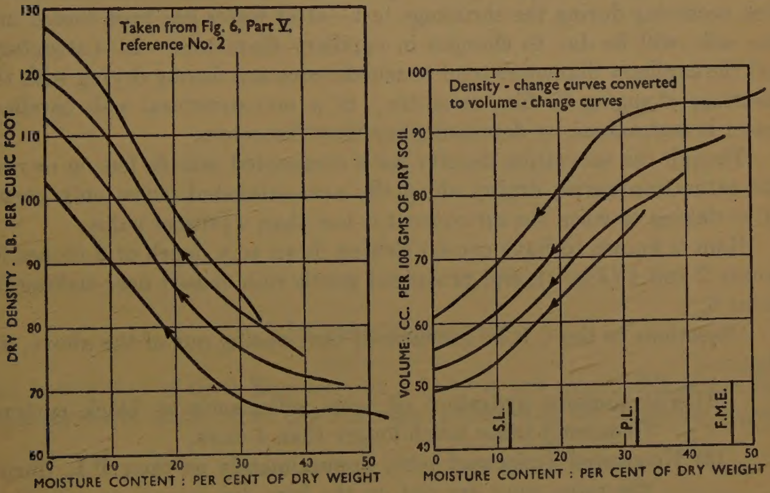
The greater the percentage compaction, the greater the apparent volume-change on drying. Porter,[2] however, concluded that there was not a great difference, and it is likely to be even less in the field.

The swelling decreases with increased compaction moisture content up to a critical value, when it approaches zero at a moisture content slightly in excess of optimum.

The curves show that drying curves run closely parallel to the saturation curve to approximately the P.L. (see DL in *Fig. 9* (*a*)), when the compaction moisture content exceeds a critical value, that is to say, when the entrapped air is about 3 per cent., though greater values may occur for A-5 and A-7 soils. The minimum compaction moisture content for no loss in density or absorption is slightly in excess of this value. For higher air contents, the curve changes to a sigmoid, as FBJ, showing the presence of expendable air.

On an energy analogy, 100 grammes of dry soil, represented by H, should swell to a moisture content, represented by G, exceeding that of condition D by an amount equalling the excess voids content, HL, or the additional capillary energy available.

Fig. 9 (*c*)



Extreme Form of Density-Change Curves for Type A-7 Binder Soil

Plastic limit : 32.
Plasticity index : 57.
Shrinkage limit : 12.

Field moisture equivalent : 47.
Shrinkage ratio : 1·93.
Specific gravity (on basis of S.R.) : 2·5.

Data suggest that this is approximately correct ; that is :

$$HL \rightarrow GZ \rightarrow \text{moisture difference } DZ$$

in centimetre-gramme-second units.

A dried highly compacted or remoulded sample apparently absorbs moisture in the vapour and/or film state until its osmotic imbibition is satisfied. The highly compacted soil is then saturated and there is little if any true capillary moisture. Compacted cohesive soils do not reach such high densities as represented by the S.L. and, on drying, there is entrapped and generally expendable air. The total swelling on rewetting is still apparently limited by the osmotic swelling of the near-basic shrinkage curve; that is:

$$(\text{vol. G} - \text{vol. H}) \rightarrow (\text{vol. D} - \text{vol. L})$$

Occluded air voids may gradually fill with water, (*Figs 9* (*a*) and (*b*)) or, below a critical density, may not fill until the end of this process, *Figs 9* (*c*), but in neither instance does the volume of air apparently influence the ultimate soil volume (dry) which depends only upon structure (density) and the internal swelling of the colloids. The latter instance suggests that the remaining voids act as true capillaries, filling without volume change.

Reference to capillary volume change may thus be misleading and any change during this range is probably an osmotic change, though contraction occurring during the shrinkage test—after water has been forced into the soil—will be due to changes in capillary diameters. In a structural soil the capillary diameters tend to remain constant during drying with the moisture changing to film moisture; in a non-structural soil, capillary water is maintained by decreasing capillary diameters.

Though the saturation density for a compacted sample lies on or near the saturation curve, drying along the near-saturated curve only occurs after slaking or when the air content is less than a critical value.

Rain is known to have caused slaking down to a depth of 2 inches for points 2 and 4 (*Fig. 1*), and prolonged gentle rain caused near-slaking for point 6.

Objections to the C.B.R. (saturated) test, arising out of the above, are that:—

(1) The osmotic hydration of some soils, such as black cottons, frequently takes much longer than 4 days.

(2) Uncovered desiccated soils, approximately at their P.L. during the rains, may dry out, in the long dry season, to near their S.L. Presumably the expansion requirements should be modified for such soils.

Volume-change studies indicate how the swelling pressure—or the outward effects of osmotic swelling—may vary with changes in structural conditions. In practice, the amount of desiccation appears more important to the engineer than the maximum effective pressure controlled by the amount and nature of the clay fraction.

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The Paper is accompanied by thirteen sheets of diagrams, from some of which the Figures in the text have been prepared.

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